HYNES AUDITORIUM EXPANSION
BOSTON, MASSACHUSETTS

Final Report (Volume II – Part B)
Support Documentation

Structural Calculations

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Support Documentation

Structural Calculations

CITY OF BOSTON
Kevin H. White, Mayor

BOSTON REDEVELOPMENT AUTHORITY
Robert J. Ryan, Director

PUBLIC FACILITIES DEPARTMENT
Donald B. Manson, Director

Support Documentation

Structural Calculations

Weidlinger Associates

INTRODUCTION

The attached structural calculations, details and sketches represent our working documents developed between April and December 1982 for the Expansion of the Hynes Auditorium. The calculations reflect the different options and alternatives investigated during the program development of the project. They were completed only to the degree required for establishing feasibility or practicality of the desired building program and are, therefore, incomplete. They also cover only critical aspects of the structure and do not include areas which were judged to be of standard construction without problems. Our conclusions are outlined in the structural report.

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Structural Calculations

Basic Load Assumptions

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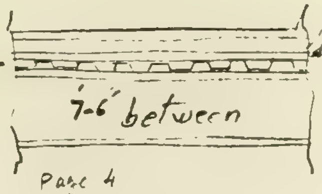
(1)

Load

(1) Roof



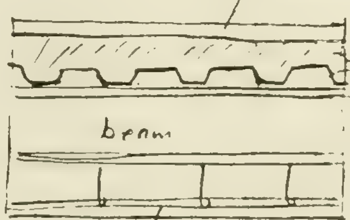
U.S.D. Type B

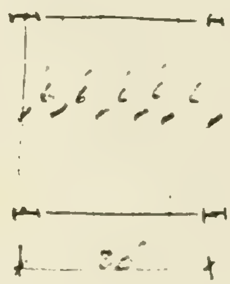
wide [illegible] uni-deck

SNOW	30	Pounds/sf
Roofing	20	"
beams	5	
	55 lb/sf	

2 = Typical floor.

(Exhibition area)





D.L = Flooring = 15

deck slab 5″ depth (page [illegible]) 50

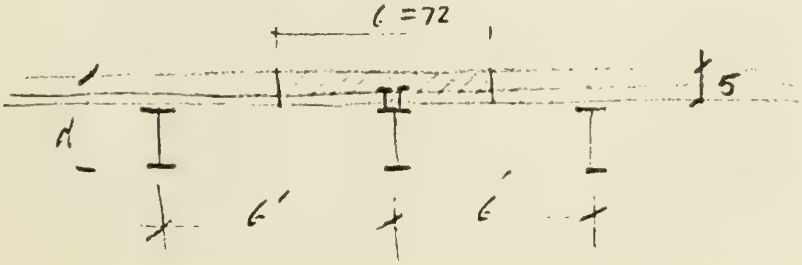
beam = 10

75 lb/sf ~~X 1.4 = 105~~

L.L = 100 lb/sf.

Fall ceiling and Mechanical = 15 } 115 lb/sf.

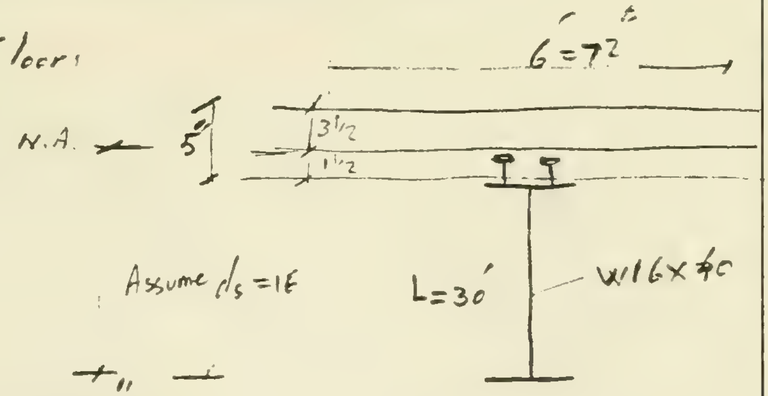
190 lb/sf.



Hynes Auditorium – Boston, Mass. Extension

2

Composite section of floors

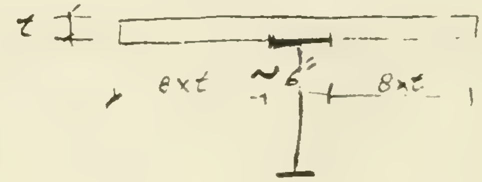


Effective b. Assume $d_s = 16$

1) $L/4 = 30/4 = 7.5' = 90''$

2) beam spacing = 72''

3) $16 \times 5 + 6'' =$ 86''



So $b = 72''$ Governs.

$$\frac{b}{n} = \frac{72''}{10} = 7.2$$

$$M = \frac{190 \times 6 \times 30 \times 30}{8} = 128.25 \text{ F.k.}$$

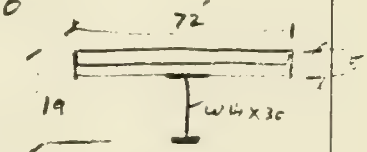
$$\sigma_v = 0.66\,F_y = 24$$

Required section modulus.

$$S_b = \frac{128.25 \times 12}{24} = 64.12^{in^3}$$

as page 144 steel design file (Composite section)

Found W 12×36 or Stronger 14×30



see page 161 : $A_s = 8.85^{in^2}$

$S_b = 67.9^{in^3}$ $S_j = 1180^{in^3}$ $S_t = 244^{in^3}$ $I = 1013^{in^4}$ $y_b = 14.72^{in}$

Stresses
- bottom fibers $= \dfrac{128.25 \times 12}{67.9} = 22.66 < 24^{ksi}$ ok
- Top fibers $= \dfrac{128.25 \times 12}{244 \times 10} = 0.63 < 0.45 F_c' = 1.35$ ok

Hynes Auditorium – Boston, Mass. Extension

(3)

Deflection

f_b = stress on bottom fibers.

$$\Delta = \frac{f_b \times L^2}{2000\, y_b} = \frac{22.66 \times 30^2}{2000 \times 14.72} = 0.69 < \frac{L}{300} = \frac{360}{300} = 1.2''$$

Horizontal shear

Shear Connectors

Conc. shear $= \frac{0.85}{2} \times f_c' \times A_c = 428.4^K$

steel shear $= \frac{1}{2} A_s f_y = 159.3^K$

use $5/8 \, \phi \, 2\frac{1}{2}''$ studs $q = 8$

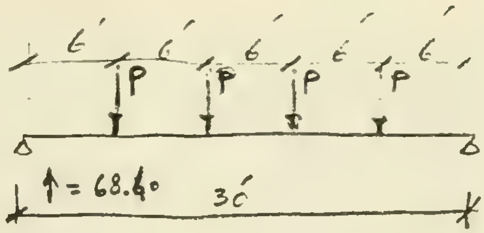
$$\frac{159.3}{8} = 19.91 \longrightarrow 20 \text{ Connectors.}$$

20 studs in one row to be welded on Top of Flange with equally distance.

4

Typical floor

Girders



$190 \times 6 = 1140$ lb.

$P = 1140 \times 30 = 34.200^{\text{Kip.}}$ $R = 68.40^{K}$ $(68.4 \times 15 - 34.2 \times 12)$

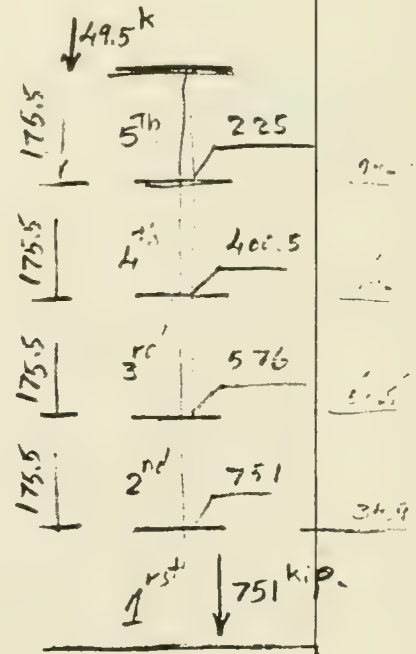
$M = P(6+12) = 615.6$ Kip-Ft.

select W 30 × 116 with Allowable moment 658 F.k.

Total Load of each interior Column.

Roof. $\frac{30 \times 30 \times 55}{1000} = 49.5$ K

Typical floor $\frac{30 \times 30}{1000} \times 195 = 175.$ k.

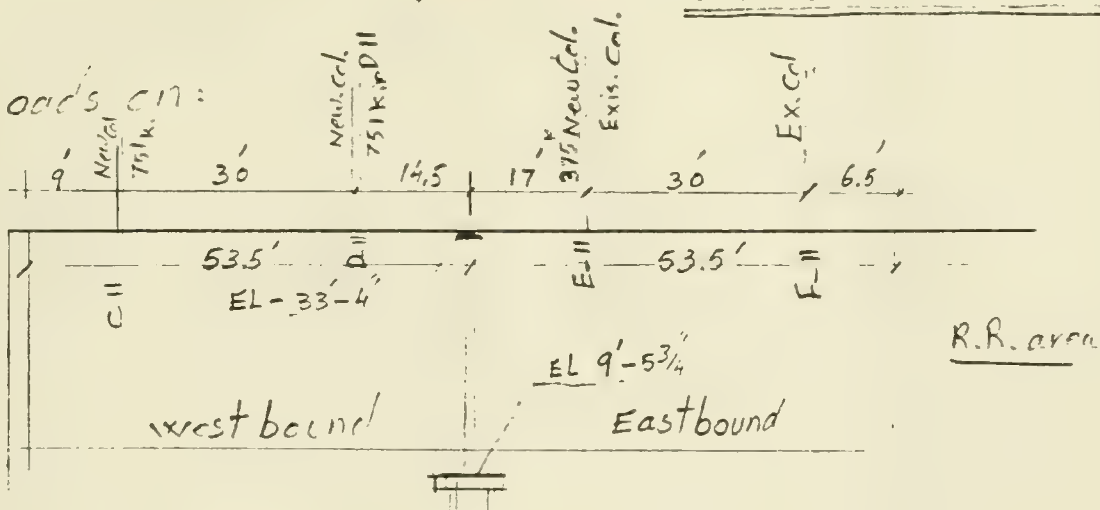


Hynes Auditorium Boston, Mass. Extension

(5)

Checking Turnpike's column and its piles on
Axies 11 & HD as F15 plan 14WF 219 1000 kips

Loads on:



5 pipe pile in 120 Ton Each

HD 11

From C11

$\frac{751 \times 9}{53.5} = 126.34^{k}$

From D11

$\frac{751 \times 39}{53.5} = 547.46$

additional Load 673.80 k
Existing load 1000

1674 Kip
256
1930

From col. on E-11

$\frac{375 \times 36.5}{53.5} = 255.80$ Kip.

256 k

length of column $33.33' - 9.48 = 23.85' \to 24'$

see page 3-13 Fy 36 W 14 x 219 for a Height of 24 Ft is allowable 1054 Axial load < 1930 K

Piles as F-1 plan

$5 \times 120 \times 2 = 1200 \text{ K} < 1930 \text{ K}$

Hynes Auditorium Boston, MA. Extension

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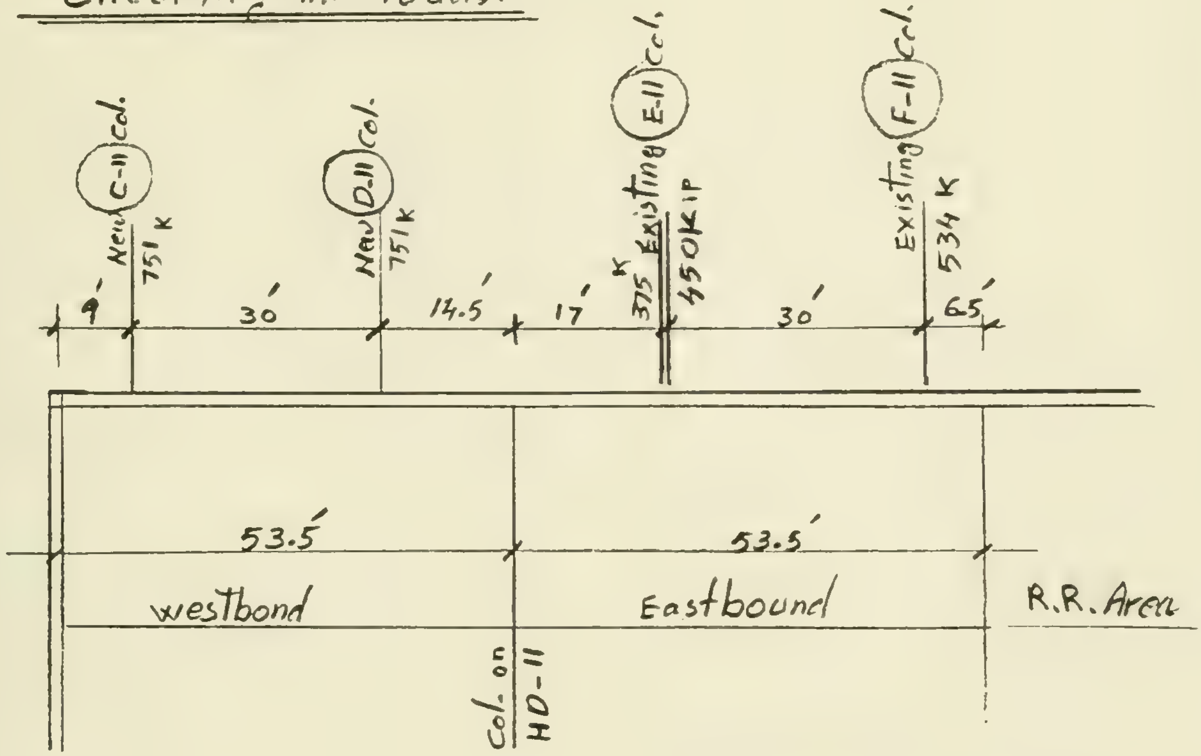
Column HD-11

checking the loads.



Hynes Auditorium, Boston, MA. Extension

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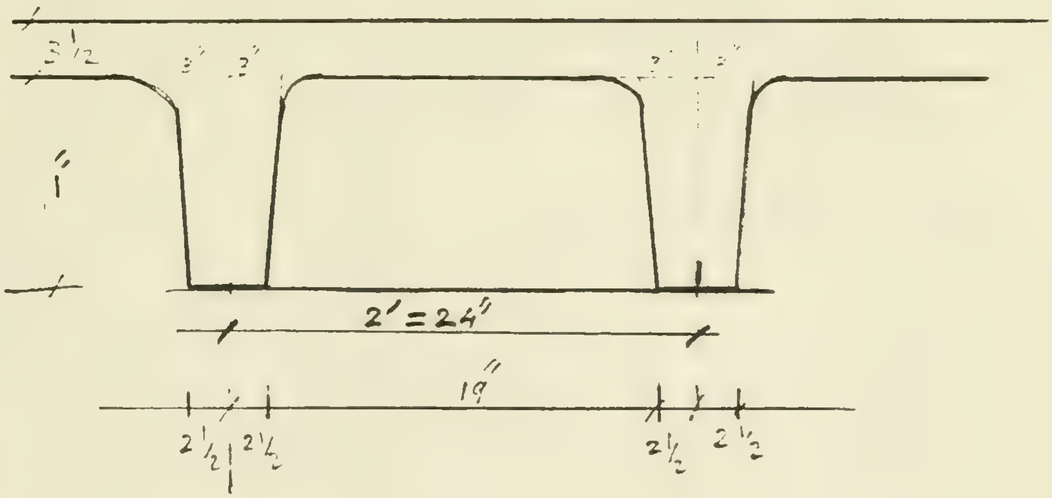
Second Floor slab. (Existing)



weight of Two ways Ribed slab. for a part $2' \times 2' = 4^{SF.}$

slab $= 2' \times 2' \times 0.2917' \times 150^{lb} = 175.0\cancel{2}$

Ribs $= \left[\left(4 \times \frac{2.5'' + 3''}{2} \times 19'' + 2.5 \times 1'\right) \div 144\right] 150 = 246.35$

421.35

$\frac{421.35}{4} = 105.30$ lb/sf.

add 10% for beam $105.30 \times 1.10 = 116$ lb/sf.

D.L =	slab	116
	Finishing floor	15
	Other	9
		140 lb/sf

L.L =	Exhibition area	100 lb/sf
	Ceiling and Mechanic	20
		120

Total Load $140 - 120 = 260$ lb/sf

Hynes Auditorium, Boston, MA Extension

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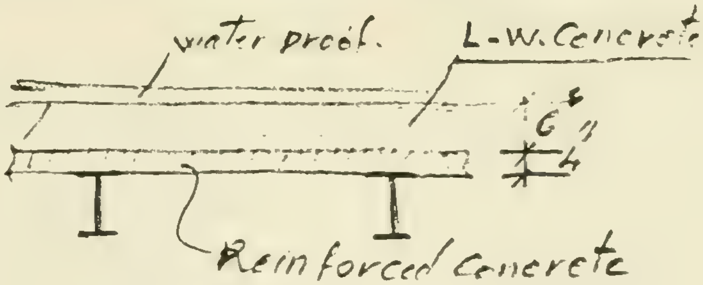
Roofing
(Existing)



D.L

Conc. slab	50
sloping	35
water proofing	5
	90 lb/sf

L.L

snow	30
Fall ceiling and Mechan.	15
	45

Total Load per square foot 135 lb/sf

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DATE
BY
NO. 9
OF

(9)

Total load of Col. H5-11

From Col C-11 $\frac{225 \times 9}{53.5} = 37.85^k$

" " D-11 $\frac{225 \times 39}{53.5} = 164.09$ k

" " E-11 $\frac{(112+239) \times 36.5}{53.5} = 239.46$ k

" " F-11 $\frac{234 \times 6.5}{53.5} = 28.43^k$

From main floor $\frac{2 \times 53.5 \times 30 \times 320}{2} = 513.60^k$

$= 983.43$ k

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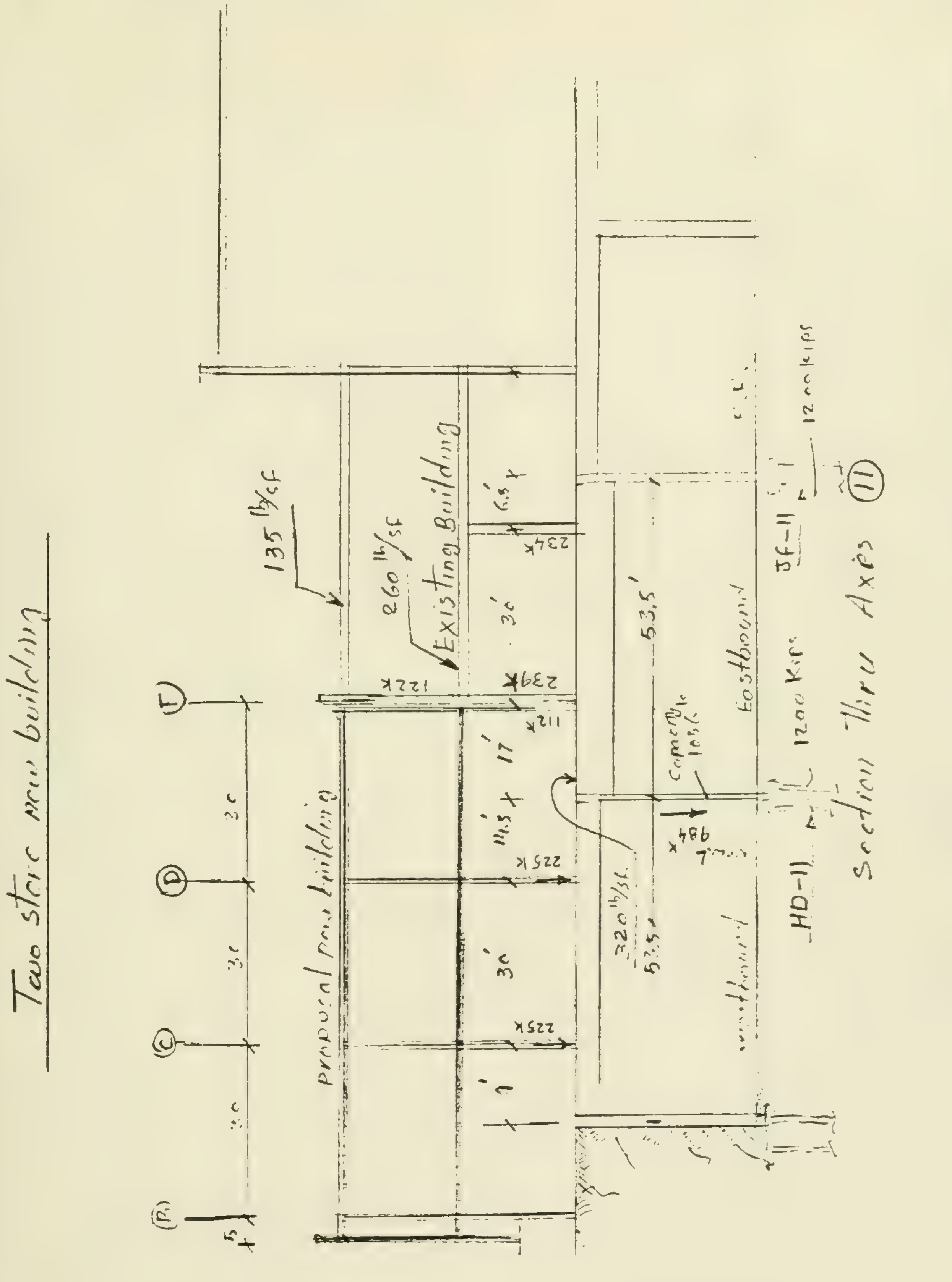
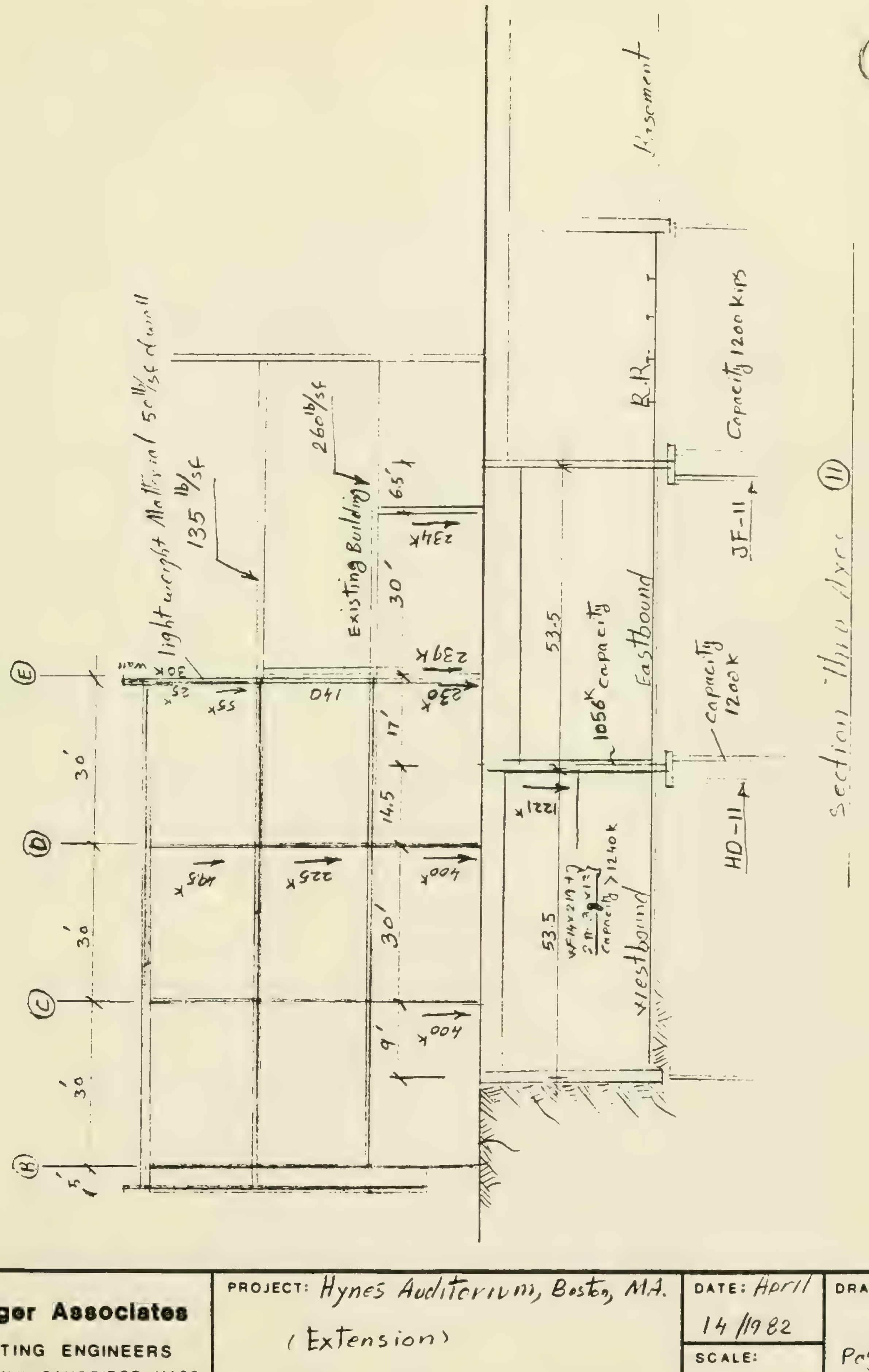
DATE
BY
NO.
OF

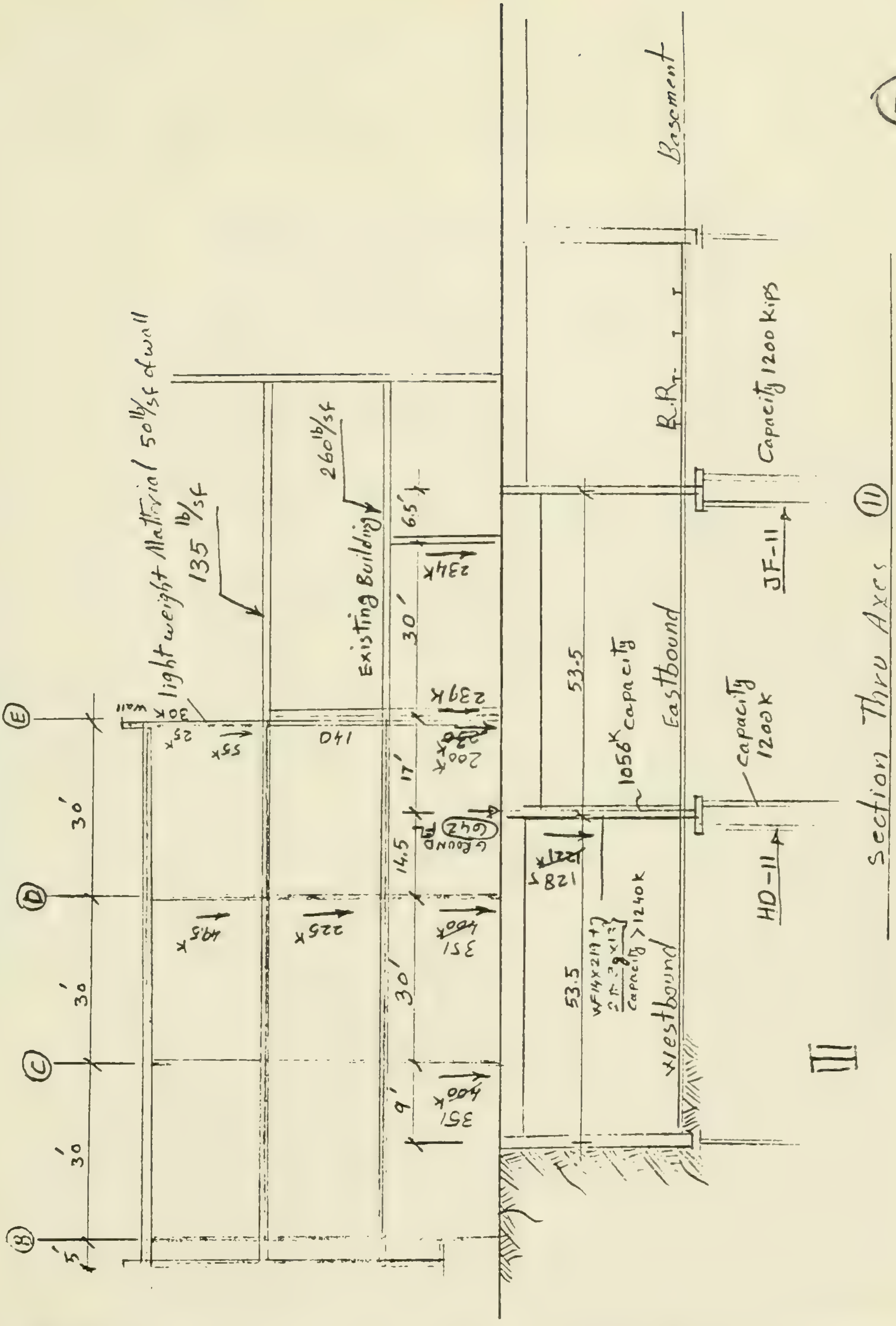
Hynes Auditorium, Boston, Ma. Extension

(10)





Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium, Boston, MA. (Extension) Three Store new building	DATE: April 14/1982 SCALE:	DRAWING NO. Page 11



Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium, Boston, MA. (Extension) Three Store new building	DATE: April 14/1982 SCALE:	DRAWING NC Page 11

(13)

Total load on column HD-11

From Column C-11 $\frac{400 \times 9}{53.5} = 67.29$ K

" " D-11 $\frac{400 \times 39}{53.5} = 291.59$

" " E-11 $\frac{(230+239) \times 36.5}{53.5} = 319.97$

" " F-11 $\frac{234 \times 6.5}{53.5} = 28.43$

" main floor $53.5 \times 30 \times \cancel{320}$ 400 $= 513.60$

1220.88 K.

Column capacity 1056 K

WF 14x219 + 2 plate 3/8 x 13 Area = 64.4x9.75 = 74.15

Capacity = 1240K

Structural Calculations

Structure Along Boylston Street

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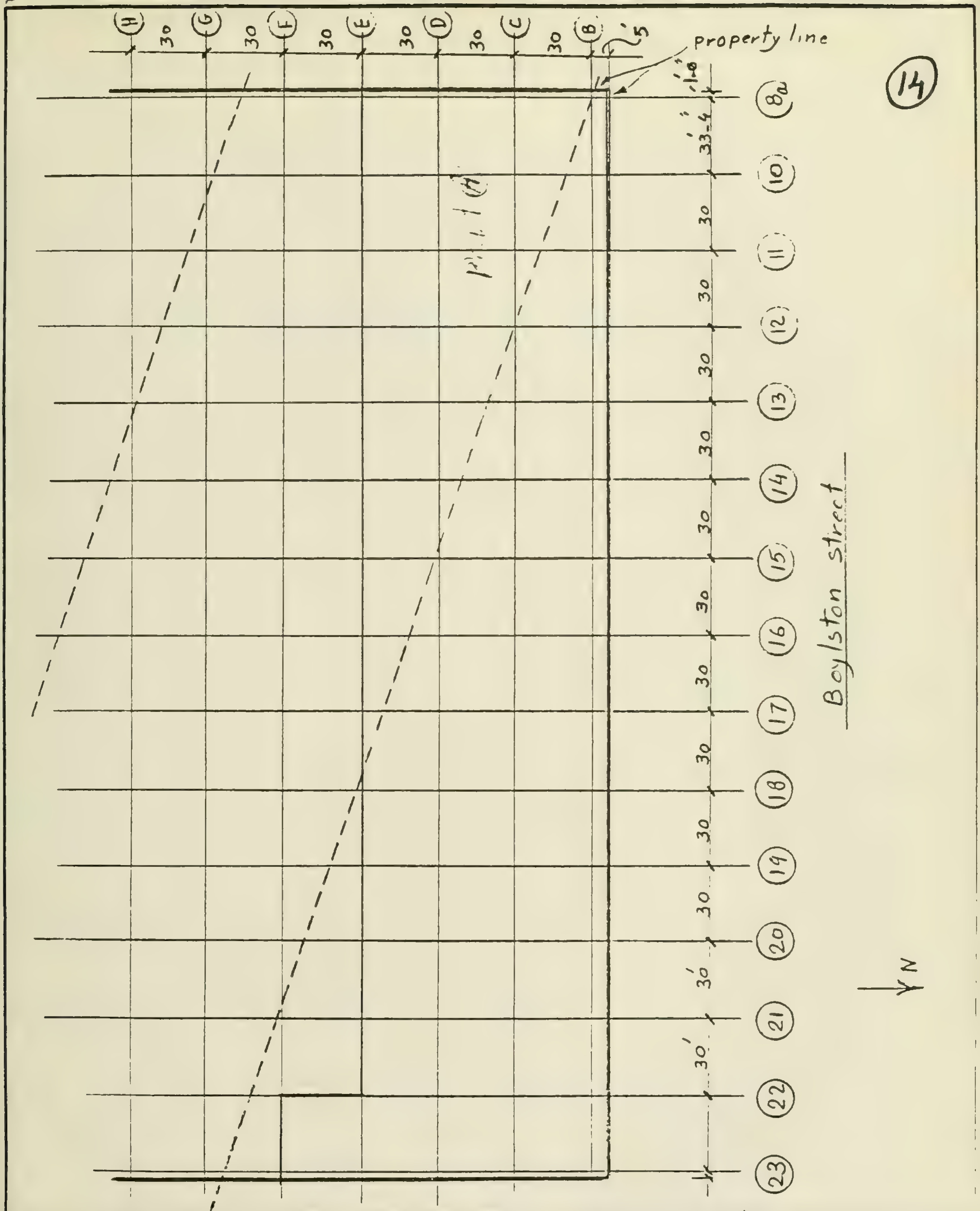
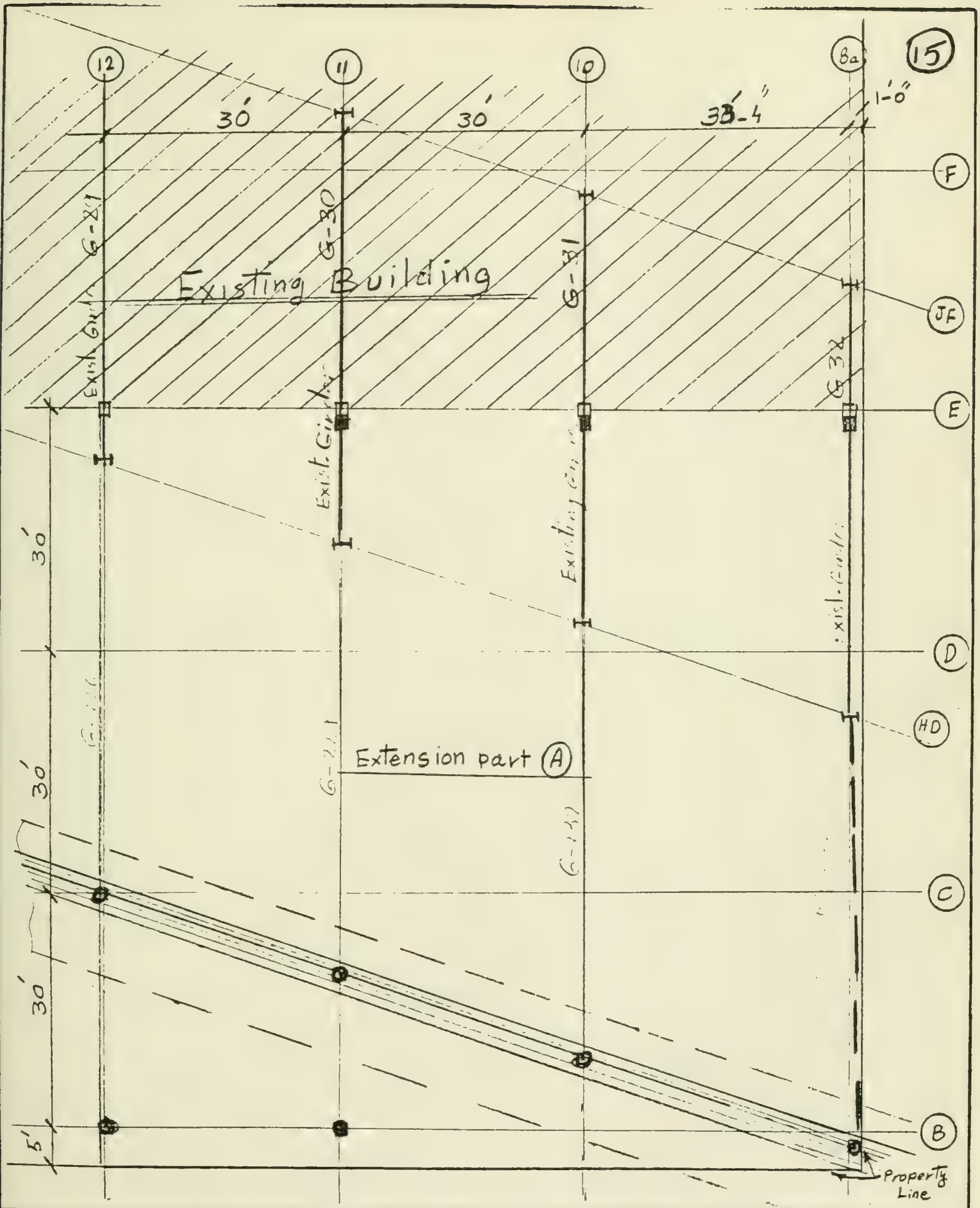


Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston, MA. Extension part. A	DATE: April 12/1982 SCALE: 1" = 50'	DRAWING NO. Sitution plan M.B.



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PROJECT: Hynes Auditorium, Boston

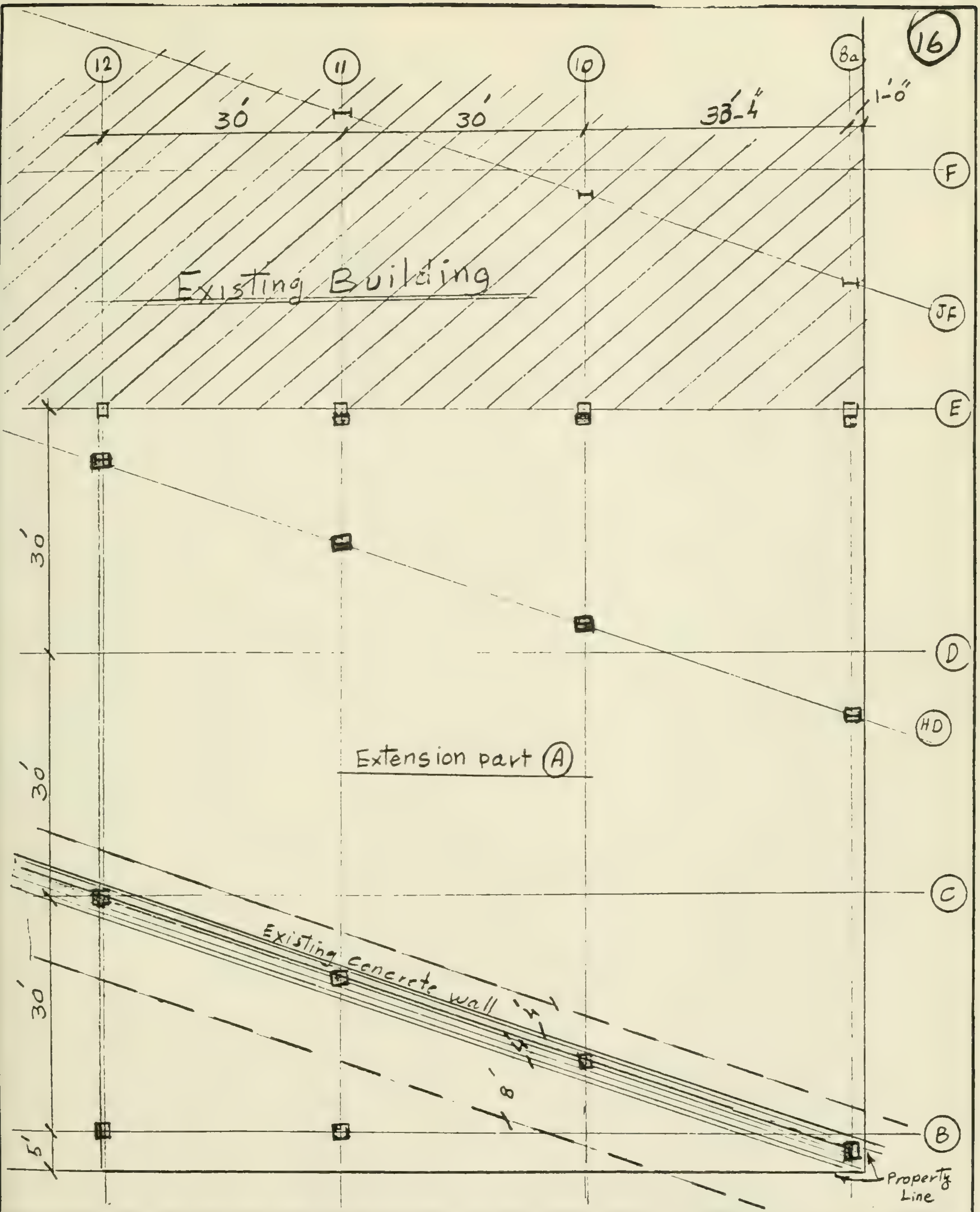
Extension part A
Ground Floor

DATE: April 15/88

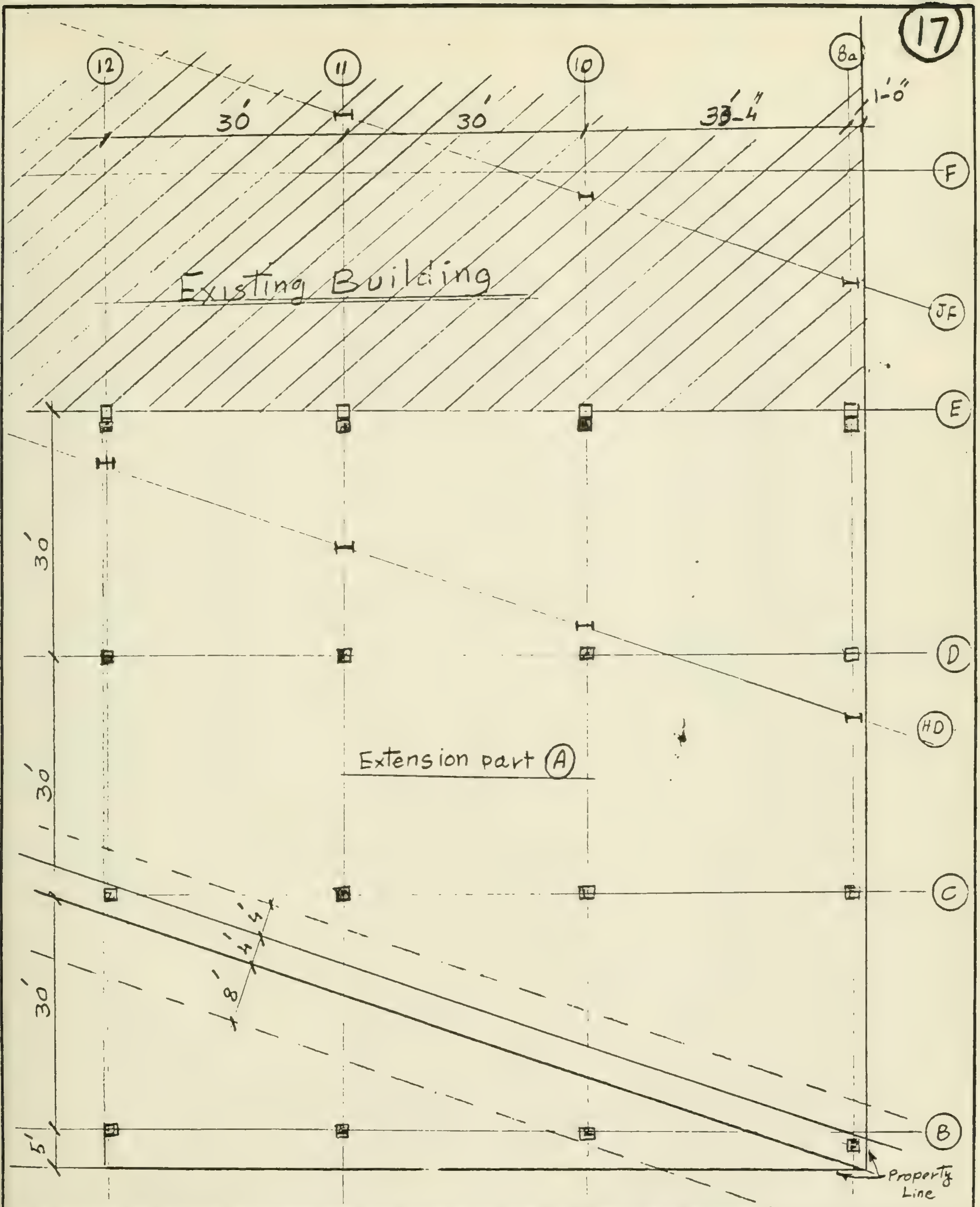
SCALE: 1"=16'

DRAWING NO. Page I

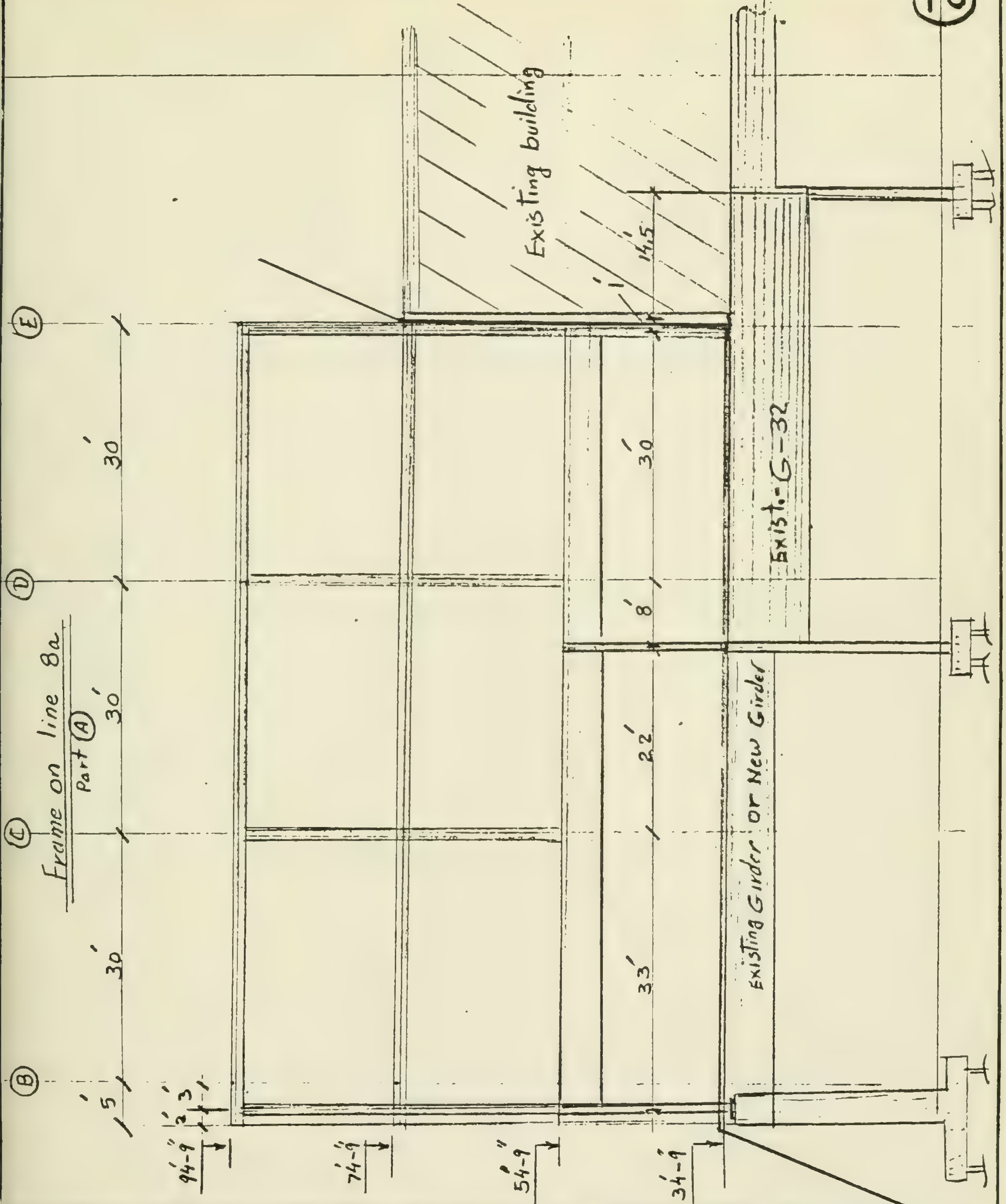
M.B



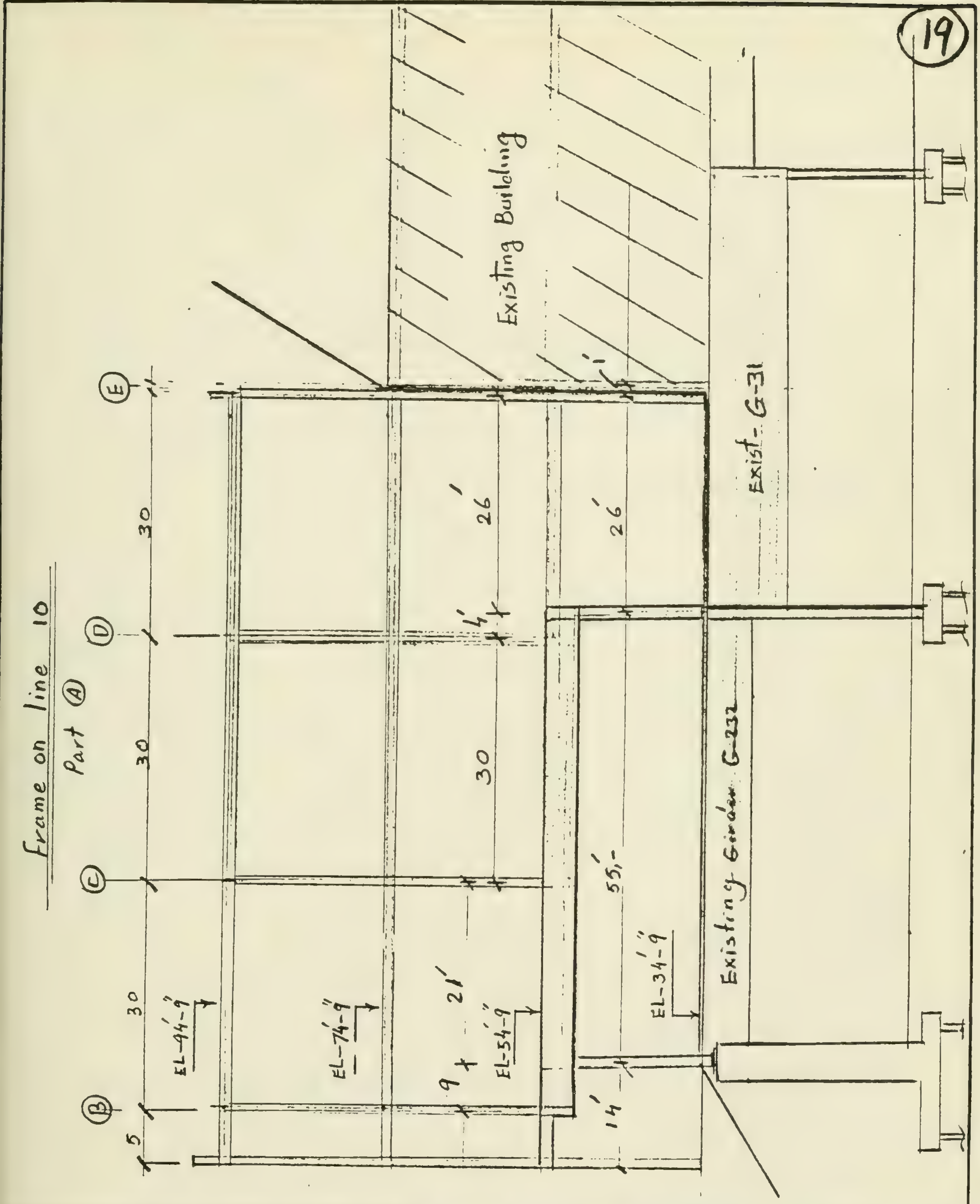
Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston Extension Part A Ground Floor (New [illegible]	DATE: April 15/82 SCALE: 1"=16'	DRAWING NO. Page 2 M.B



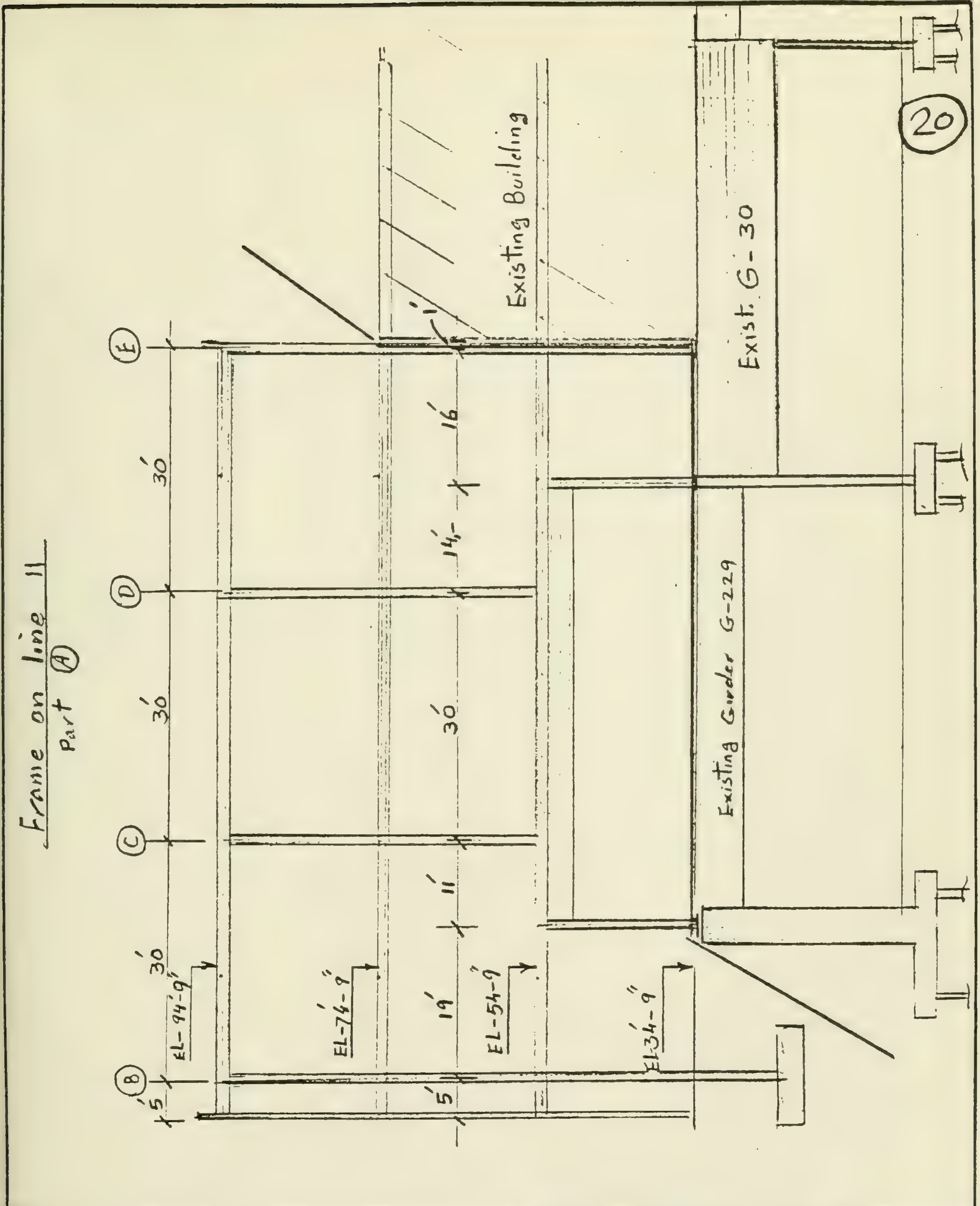
Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Audilorium Boston Extension part (A) Typical Floor (Columns plan)	DATE: April 19/82 SCALE: 1" = 16'	DRAWING NO. Pr-: (3) M.E.



Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Extension (part A)	DATE: April 19/82 SCALE: 1" = 16'	DRAWING NO. Page 4 M.B



Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston Extension Part (A)	DATE: April 19/82 SCALE: 1" = 16'	DRAWING NO. Page 5 M.B



Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston Extension Part (A)	DATE: April 19/82 SCALE: 1" = 16'	DRAWING NO. Page 6 M.B

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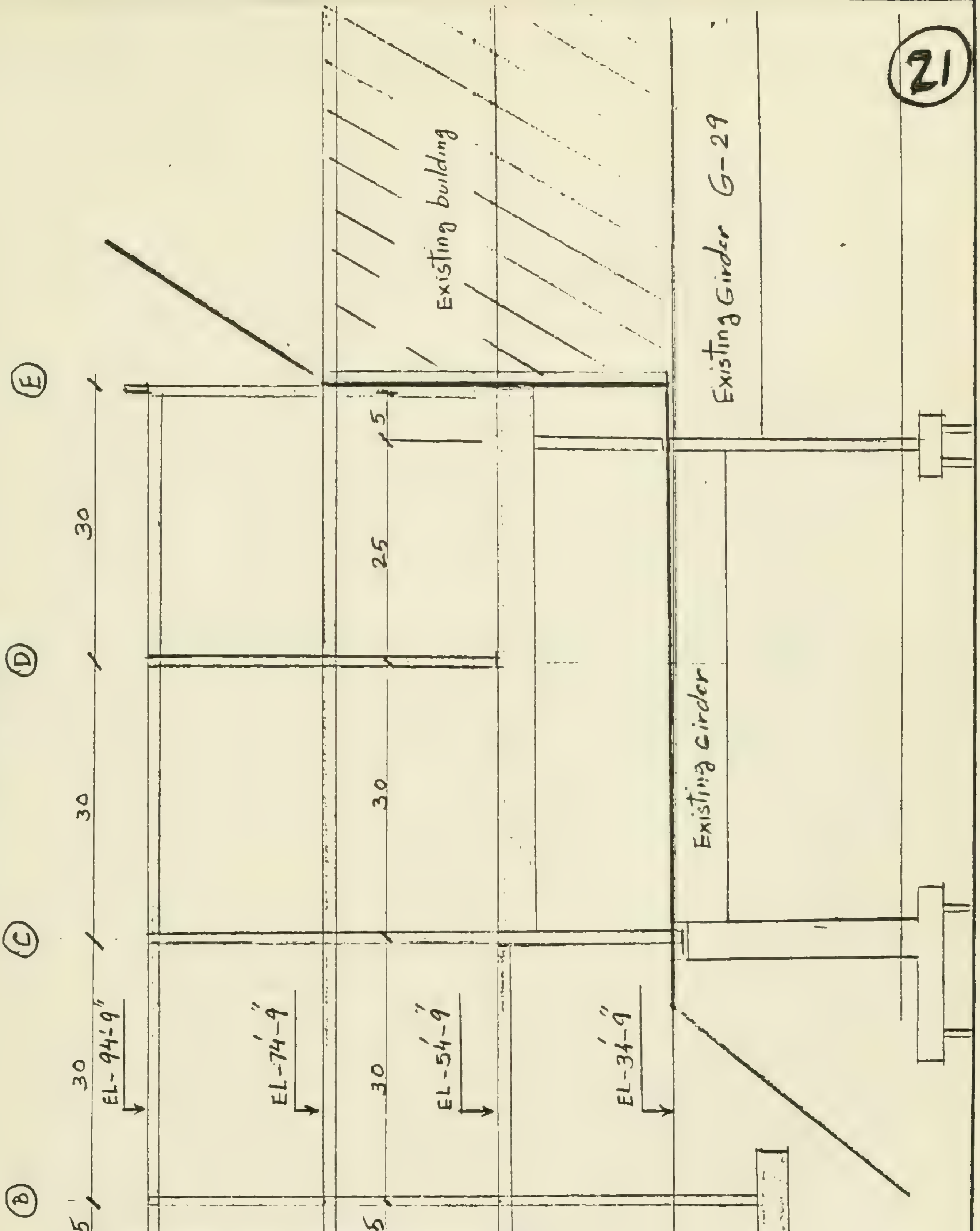
Frame on line 12
Part (A)



Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston Extension part (A)	DATE: April 19/82 SCALE: 1" = 16'	DRAWING NO. Page 7 M.B

Loads:

a) Roof: used light weight Concrete and 1½ B-lok Deck

weight of Concrete and Deck	36	lb/SF.
" " Beams	10	"
" " ceiling and mechanical	15	"
" " water proofing and Gravel	15	"
" " Snow	30	"
Total weight of Roofing	106	P/SF

Assume 110 P/SF.

b) Typical floor.

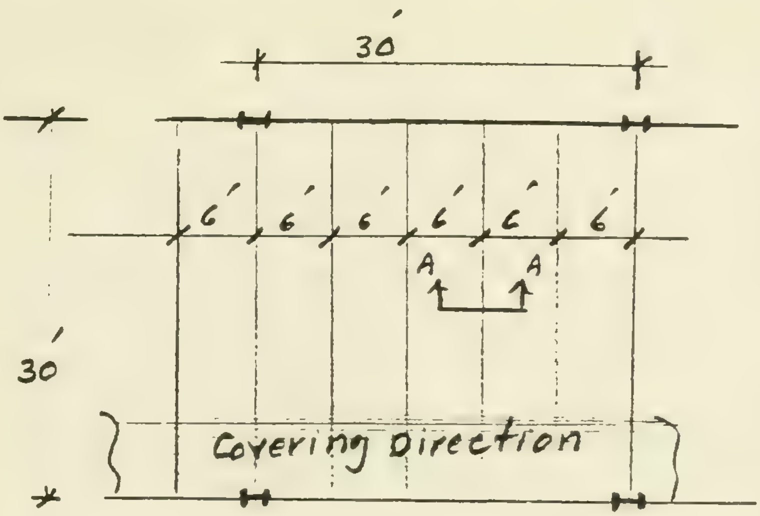
weight of Concrete and Deck	36	lb/SF
" " Beam	10	"
" " Ceiling + Mechanical	15	"
" " L-L	100	
Total weight of Typ. Floor's slab.	161	P/SF.

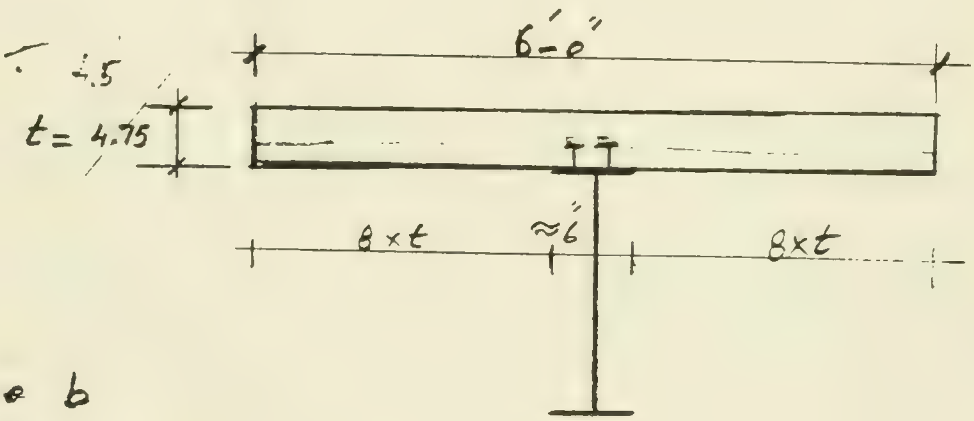
Assume 160 P/SF.

because L.L is 100 P/SF for separation wall · No consider any weight

For side walls assume 30 P/SF and for 20' hight 600 P/F linear

Hynes Auditorium, Boston, MA. Extension part (A)





Effective b

1) $L/4 = \frac{30}{4} = 7.5'$

2) Beam spacing = 6' = 7

3) $16 \times 4.75 + 6 = 6'\text{-}10'' = 7$

Govern $b = 6' = 72''$

24

Roof framing design

f'_c of light weight concrete = 3000

f_y (w.w.) = 60000

$n = 10$

(1) Beams.

$M = \frac{6 \times 10 \times 30^2}{8} = 74.22$ F-k.

$\frac{b}{n} = \frac{72^2}{10} = 7.2$

$f_y = 36000^{\#/\text{in}}$ $\sigma_u = 24$

S_b = Required section modulus $= \frac{74.22 \times 12}{22} = 40.48$

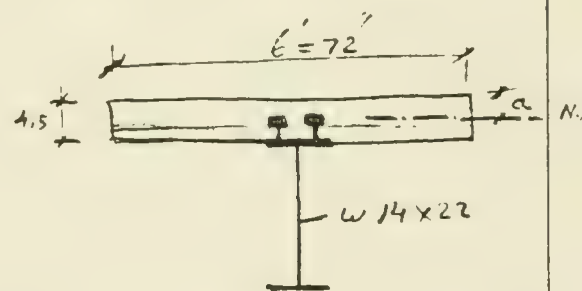
see page 19 steel design file (Bethelehem)

for 4.5" slab Thikness

For $\frac{b}{n} = 7.2$ $S_b = 37.11$ will find W 14×22

see page 45

For $n = 10$ $\frac{b}{n} = 7.2$



$S_b = 49.0^{\text{in}^3}$ $S_t = 191^{\text{in}^3}$ $I = 711^{\text{in}^4}$ $Y_b = 14.51$ $A_s = 6.49^{\text{in}^2}$

$M_r = 98$ Ft-k, $V_h = 117^k$ $C_h = 245^k$

stresses
- bottom fiber $= \frac{74.22 \times 12}{49} = 18.17 < 22^{\text{k}/\text{in}^2}$ ok
- Top fiber $= \frac{74.22 \times 12}{191 \times 10} = 0.466 < 0.45 f'_c = 1.35$ ok

$a = \frac{A_s \times F_y}{0.85 f'_c b} = \frac{6.49 \times 36}{0.85 \times 3 \times 72} = 1.27^{\text{in}}$

25

Diflection

$$\Delta = \frac{f_b \times L^2}{2000 \times Y_b} < \frac{L}{300}$$

f_b = stress on bottom fibers.

$$\Delta = \frac{18.17 \times 30^2}{2000 \times 14.51} = 0.56 < \frac{360}{300} = 1.2''$$

ok

Horizontal shear : shear Connectors

$$\text{conc. shear} = \frac{0.85 f'_c \times A_c}{2} = \frac{0.85 \times 3 \times 72 \times 4.5}{2} = 413 \text{ Kip}$$

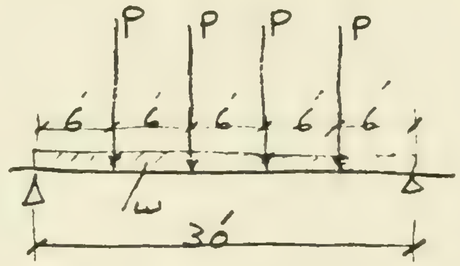
$$\text{steel shear} = \tfrac{1}{2} A_s \times F_y = \frac{6.49 \times 36}{2} = 116.82$$

Use $5/8'' \phi \; 2\frac{1}{2}$ stude $q = 8$

$$\frac{116.82}{8} = 14.60 \longrightarrow 16 \text{ stud}$$

Roof Girders

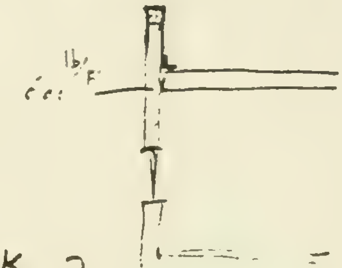
(a) Edge Girders (Composite)



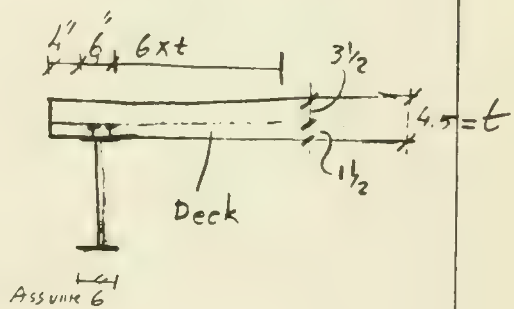
Partial slab

$P = 110 \times 6 \times 15 = 9.9$ Kips

$w = 600$ lb/F linear



$M_p = 9.9 \times 12 + 9.9 \times 6 = 178.7$ F.K.
$M_w = \frac{0.600 \times 900}{8} = 67.5$ " "
246.2 F.K.



$b = 4'' + 6'' + 6 \times 4.5 = 37$

$\frac{b}{n} = \frac{37}{10} = 3.7 \rightarrow 4$

Required section modulus is

$$S = \frac{246.2 \times 12}{22} = 134.29 \text{ k}$$

See chart page 10.1 of steeldesign file (Properties of Composite.

For $b/h = 3.7$ and $S = 134.29$ W 18 x 55 is ok but 24 x 55 W21X57 is better

$S_b = 148 \text{ in}^3$ $S_J = 678 \text{ in}^3$ $S_t = 307 \text{ in}^3$ $I = 2534 \text{ in}^4$ $y_b = 17.06$ in $A_s = 16.19 \text{ in}^2$

stress on
- bottom fiber steel $= \frac{246.2 \times 12}{148} = 19.96 < 22$ K/in² ok
- Top Fiber Concrete $= \frac{246.2 \times 12}{307 \times 10} = 0.96 < 0.45 f'_c = 1.35$ ok

(27)

Deflection $\Delta = \frac{f_b \times L^2}{2000 \times y_b} \leq \frac{L}{300}$

$\frac{L}{300} = \frac{360}{300} = 1.2''$ f_b = stress on bottom fibers.

$\Delta = \frac{19.96 \times 30^2}{2000 \times 17.06} = 0.52 < 1.2''$ ok

Horizontal shear : shear Connector

Shear of Concrete $= \frac{0.85 \times f'_c \times A_c}{2} = \frac{0.85 \times 3 \times 37 \times 4.5}{2} = 212.29^K$

" " steel $= \frac{A_s \times F_y}{2} = \frac{16.19 \times 36}{2} = 291.42$

Use $\frac{5}{8}\phi$ $2\frac{1}{2}''$ stud $q = 8$

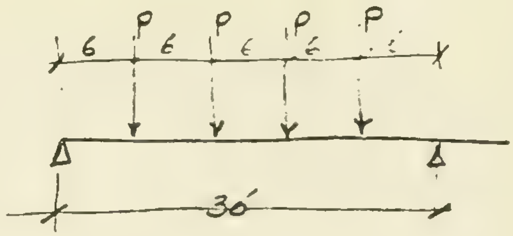
$N = \frac{212.29}{8} = 26.53 \longrightarrow$ 30 stud in half beam

2 stud each foot with equal destence.

(28)

Roof Girders

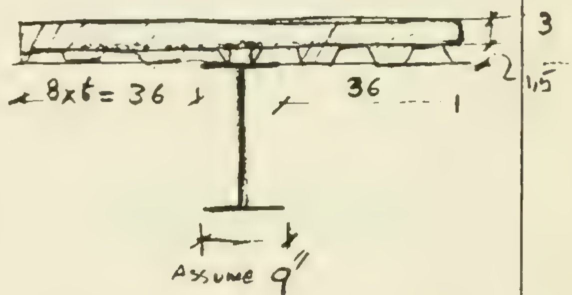
(b) Interior Girders.



$P = 110 \times 6 \times 30 = 19.8$ kips.

$M_{max} = 19.8 \times 12 + 19.8 \times 6 = 357.4$ f.k

$b = 36 + 9 + 36 = 81^{in}$



$n = 10$ $f'_c = 3000$

$b/n = 8.1 \rightarrow 8$

Required section modulus $= \dfrac{357.4 \times 12}{22} = 194.94\ in^3$

W24-84

W 24×76

See page 32 (DESIGN Data: properties of Composite – (section with steel deck – Bethlehem)

$S_b = 238\ in^2$ $S_t = 491$ $I = 4560$ $y_b = 19.13$

Stress on:
$$\begin{cases} \text{bottom fibers} = \dfrac{357.4 \times 12}{238} = 18 < 22^k \text{ ok} \\ \text{Top Fibers} = \dfrac{357.4 \times 12}{491 \times 10} = 0.87 < 0.45 f'_c = 1.35^k \text{ ok} \end{cases}$$

Δ = Deflection $= \dfrac{F_b \times L^2}{2000 \times y_b} \leq \dfrac{L}{300} = \dfrac{360}{300} = 1.2$ in

29

$$\Delta = \frac{18 \times 30^2}{2000 \times 19.13} = 0.42 < 1.2'' \quad \text{ok}$$

Horizontal shear

Shear Connector

$$\text{Shear of Concrete} = \frac{0.85 \times f'_c \times A_c}{2} = \frac{0.85 \times 3 \times 81 \times 45}{2}$$

horizontal shear of concrete = 464.74 k

$$\text{horizontal shear of steel} = \frac{A_s \times F_y}{2} = \frac{16.19 \times 36}{2} = 291.42\ k$$

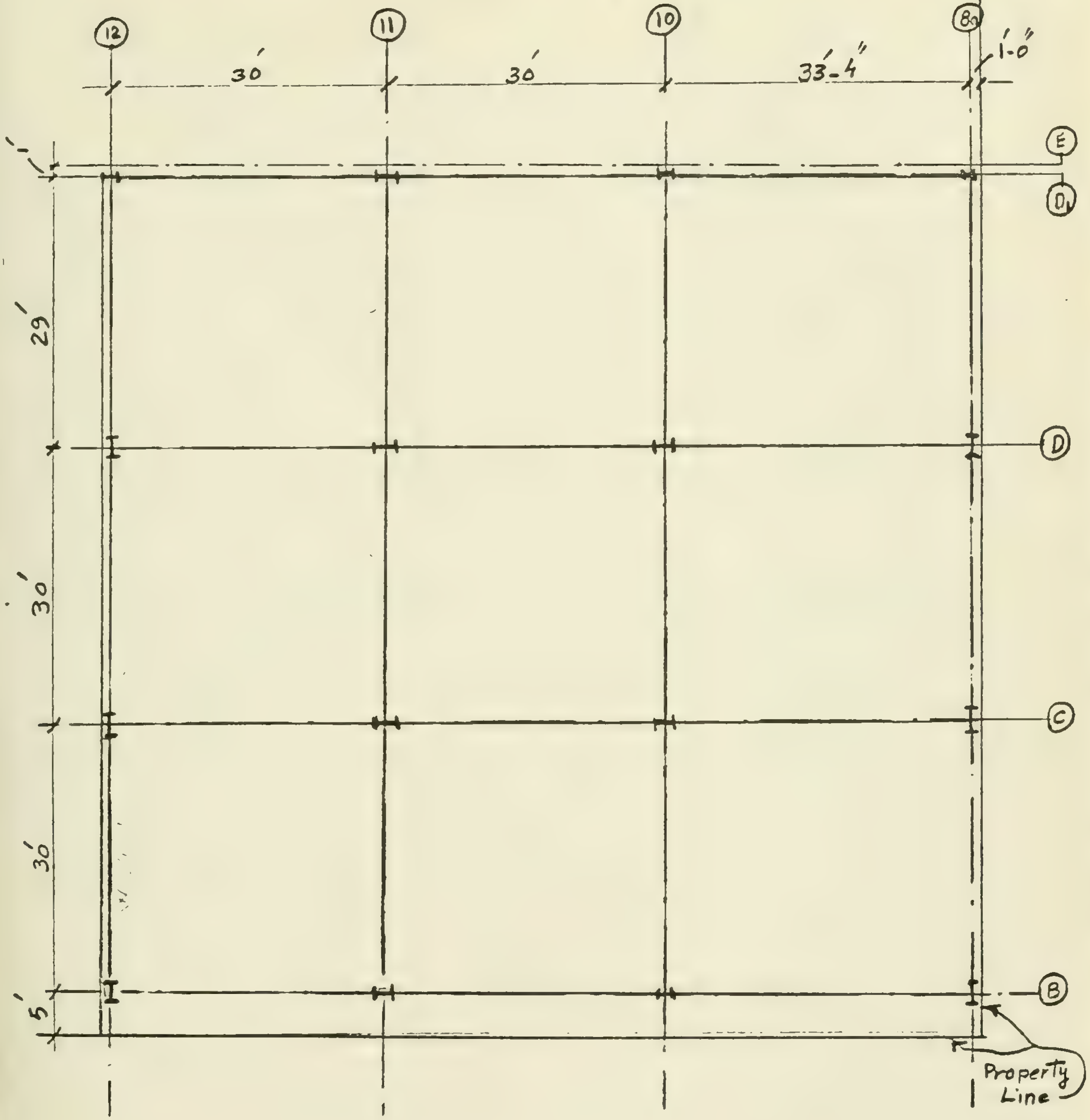
use 5/8 ϕ 2½" stud $q = 8\ k$

$$N = \frac{291.42}{8} = 36.43 \rightarrow 38$$

2 row Each 19 stud with equal distance

30

See page 10 → 13



Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston Extension Part (A) Columns plan	DATE: April 20/1982 SCALE: 1" = 16'	DRAWING NO. Page 16 M.B.

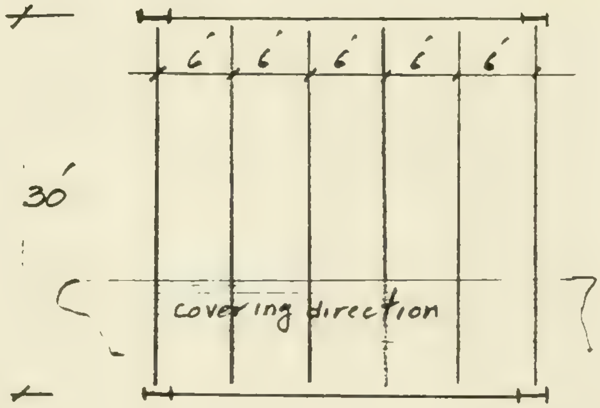
(31)

Second floor

beams

Effective $b = 6' = 72''$

(See page 2)



$$M = \frac{160 \times 6 \times 30^2}{8} = 108 \text{ F-k.} \qquad \frac{b}{n} = 7.2$$

$$S_b = \text{Required section modulus} = \frac{108 \times 12}{22} = 58.90 \text{ in}^3$$

see page 20 selection chart for steel section

"W 16x26" see page 42 $\frac{b}{n} = 7.2 \longrightarrow 8$

$S_b = 62.7 \text{ in}^3 \quad S_t = 230 \text{ in}^3 \quad I = 995 \text{ in}^4 \quad y_b = 15.86 \text{ in} \quad A_s = 7.68 \text{ in}^2$

$$\text{stresses}\begin{cases} \text{bottom fiber} = \dfrac{108 \times 12}{62.7} = 20.67 < 22 \text{ k/in}^2 & \text{ok} \\ \text{Top fiber} = \dfrac{108 \times 12}{230 \cdot 10} = 0.56 < 1.35 & \text{ok} \end{cases}$$

$$a = \frac{A_s \times f_y}{0.85 \times f'_c \times b} = \frac{7.69 \times 36}{0.85 \times 3 \times 72} = 1.51 \text{ in}$$

32

Deflection

$$\Delta = \frac{20.67 \times 30^2}{2000 \times 15.86} = 0.58 < \frac{360}{300} = 1.2$$

ok

Horizontal shear : shear connecter.

$$\text{Concrete shear} = \frac{0.85 \times 3 \times 72 \times 4.5}{2} = 413.1 \text{ Kip}$$

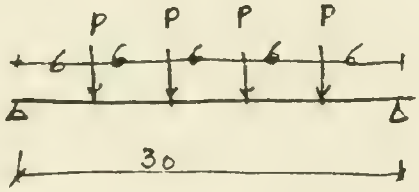
$$\text{steel shear} = \frac{7.69 \times 36}{2} = 138.42$$

use $5/8 \phi$ $2\frac{1}{2}''$ connector $q_r = 8$ kip

$$\frac{138.42}{8} = 17.30 \rightarrow 20 \text{ stud.}$$

Second Floor Girders.

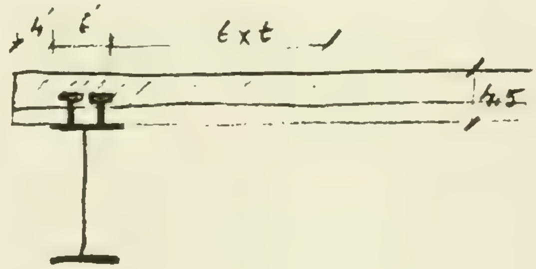
(a) Edge Girders.



Roof: $P = 160 \times 6 \times 15 = 14.4$ Kips.

Wall: $w = 600\ ^{lb}/_{F}$ linear

$$M_{max} = \begin{cases} M_p = 14.4\,(12+6) = & 259.2 \text{ F. K.} \\ M_L = \dfrac{600 \times 30^2}{8} = & 67.5 \text{ '' ''} \end{cases} = 326.7$$



$b = 4 + 6 + 6 \times 4.5 = 37''$

$\dfrac{b}{n} = \dfrac{3.7}{10} = 3.7 \rightarrow 4$

Required section modulus:

$$S = \frac{326.7 \times 12}{22} = 178.2 \ in^3$$

See chart page 22 of Design Data (1978) Bethlehem)

w 24×68

$S_b = 199\ in^3$ $\quad S_t = 296\ in^3$ $\quad I = 3360\ in^4$ $\quad y_b = 16.89\ in$ $\quad A_s = 20.10\ in^2$

$$\text{Stress} \begin{cases} \text{bottom Fibers} = \dfrac{326.7 \times 12}{199} = 19.70 < 22 \text{ K} \quad \text{ok} \\ \text{top Fibers} = \dfrac{326.7 \times 12}{296 \times 10} = 1.32 < 1.36 \text{ K} \quad \text{ok} \end{cases}$$

(34)

Deflection $\Delta = \frac{f_b \times L^2}{2000 \times Y_b} \leq \frac{L}{300} = 1.2$ $L = 30 \times 12 = 360''$

$\Delta = \frac{19.70 \times 30^2}{2000 \times 16.89} = 0.52 < 1.2$ ok

Horizontal shear: Connector.

Concrete shear $= \frac{0.85 \times f'_c \times A_c}{2} = \frac{0.85 \times 3 \times 37'' \times 4.5}{2} = 212.29^k$.

steel shear $= \frac{A_s \times f_y}{2} = \frac{20.10 \times 36}{2} = 361.80^k$

use 5/8 ϕ 2½ stude q = 8 K

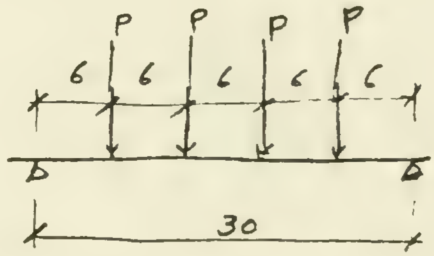
$N = \frac{361.80}{8} = 45,22 \rightarrow 46$ stude

2 Row, Each 22 stude with equal dectances

in Half of span

Second floor Girders.

(b) Interior Girders

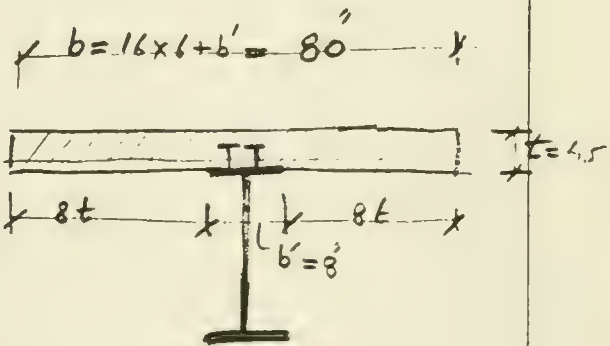


$P = 160 \times 6 \times 30 = 28.8$ Kip.

$M_{max} = 28.8 \times 18 = 518.4$ F.K.

Required section modulus $= \frac{518.4 \times 12}{22} = 282.8\ in^3$

$b = 80''$ $b/n = 8$



see : Design data properties

Page 23 select W 30×99

$S_b = 355$ $S_t = 658$ $I = 7870$ $Y_b = 22.19$ $A_s = 29.10$

stress
- on bottom fibers $= \frac{518.4 \times 12}{355} = 17.52\ k < 22\ k$ ok
- on Top Fibers $= \frac{518.4 \times 12}{658 \times 10} = 0.94 < 1.3\ k$ ok

36

Δ = Deflection $= \frac{f_b \times L^2}{2000 \times y_b} \leqslant \frac{L}{300} = \frac{360}{300} = 1.2$

$\Delta = \frac{17.52 \times 900}{2000 \times 22.19} = 0.35^{in} < 1.2$ ok

Horizontal shear (Connecters)

shear of Concrete $= \frac{1}{2}(0.85 \times f'_c \times A_c) = \frac{0.85 \times 3 \times 80 \times 4.5}{2} = 450^k$

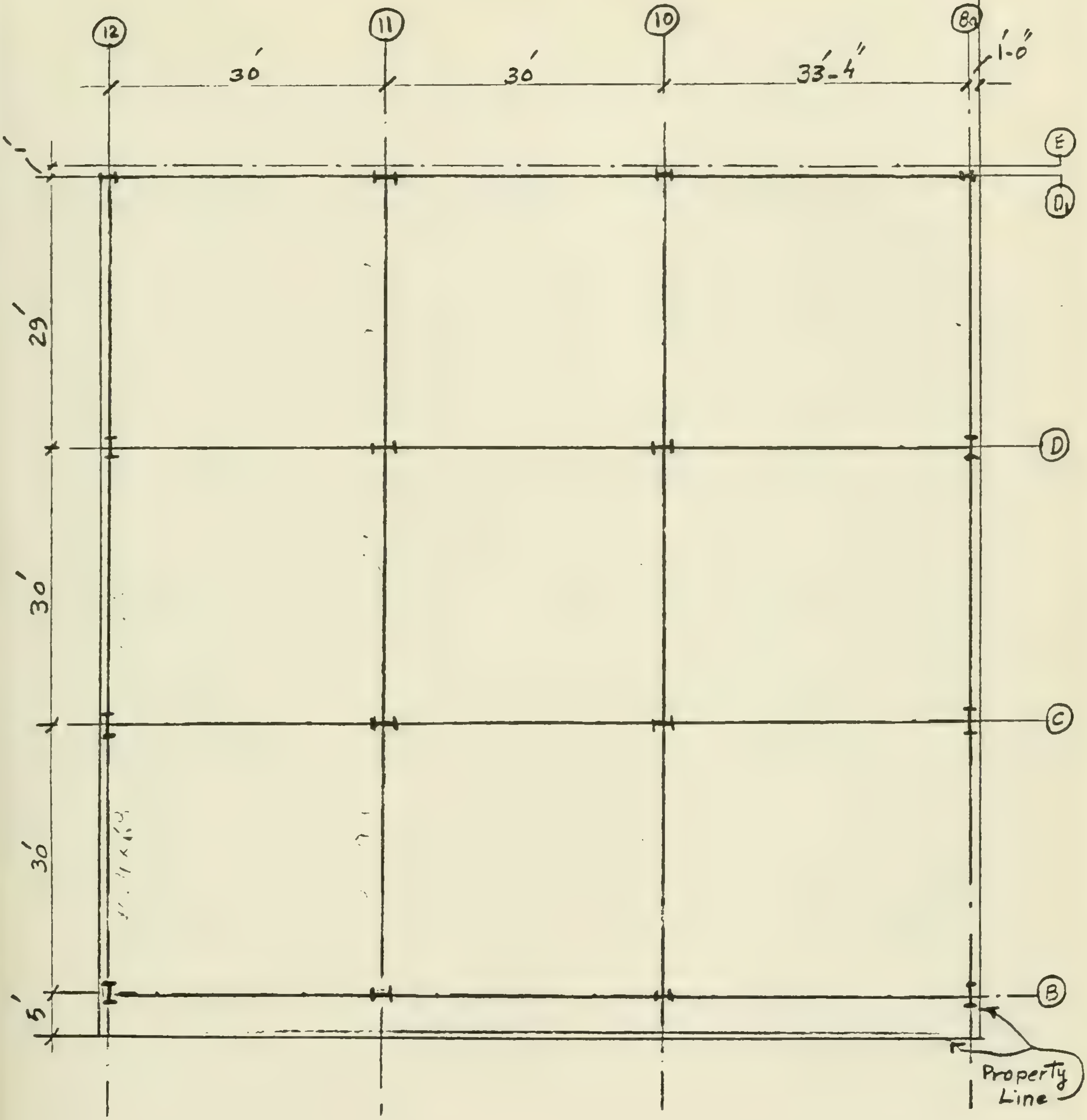
shear of steel $= \frac{1}{2} A_s \times f_y = \frac{29.10 \times 36}{2} = 523.8^k$

use $5/8\phi$ $2\frac{1}{2}''$ stud $q = 8k$

$N = \frac{459}{8} = 57.37 \rightarrow 60$ studs

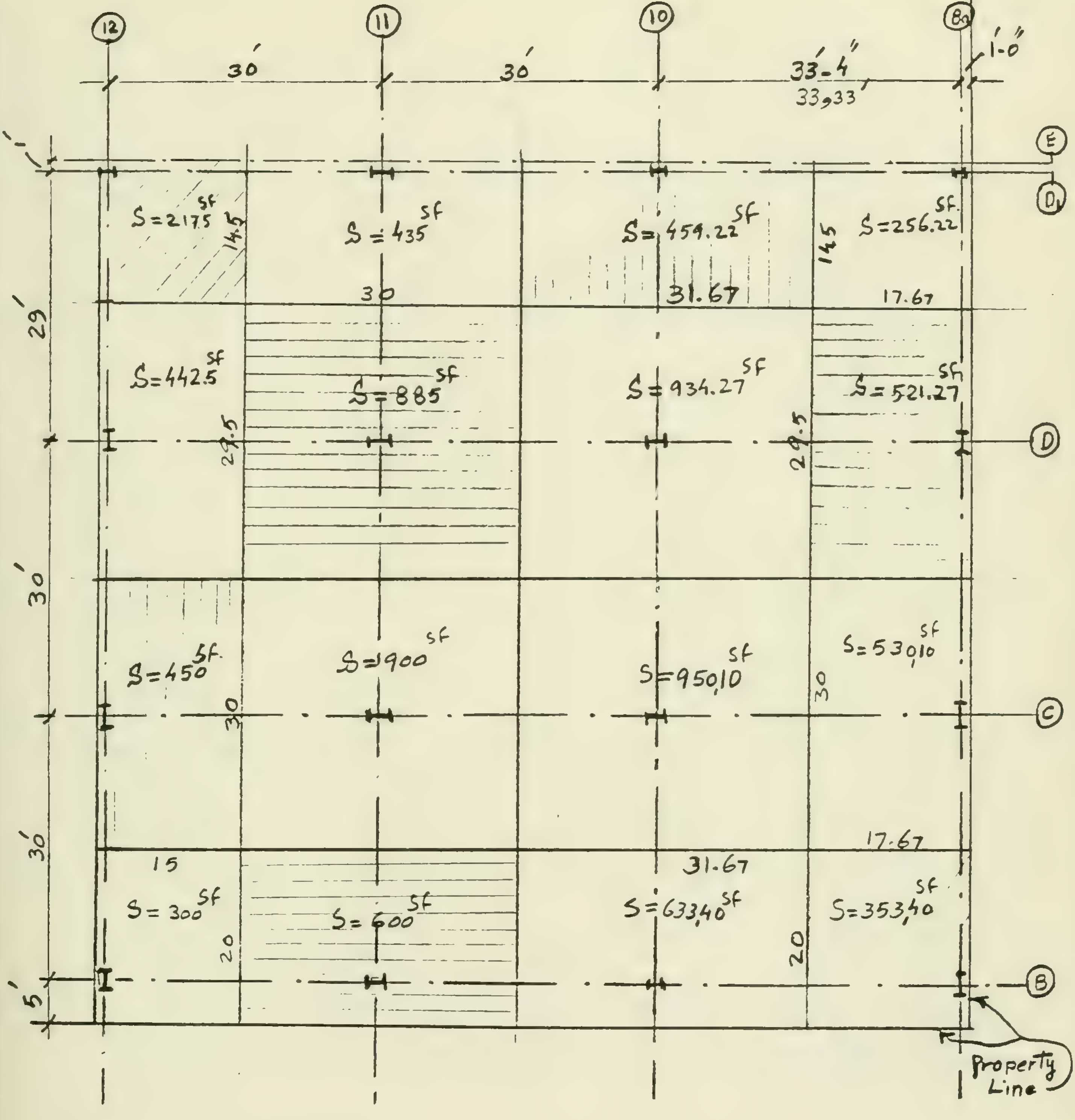
3 row, Each 20 studs with equal Distances

See page 14–17

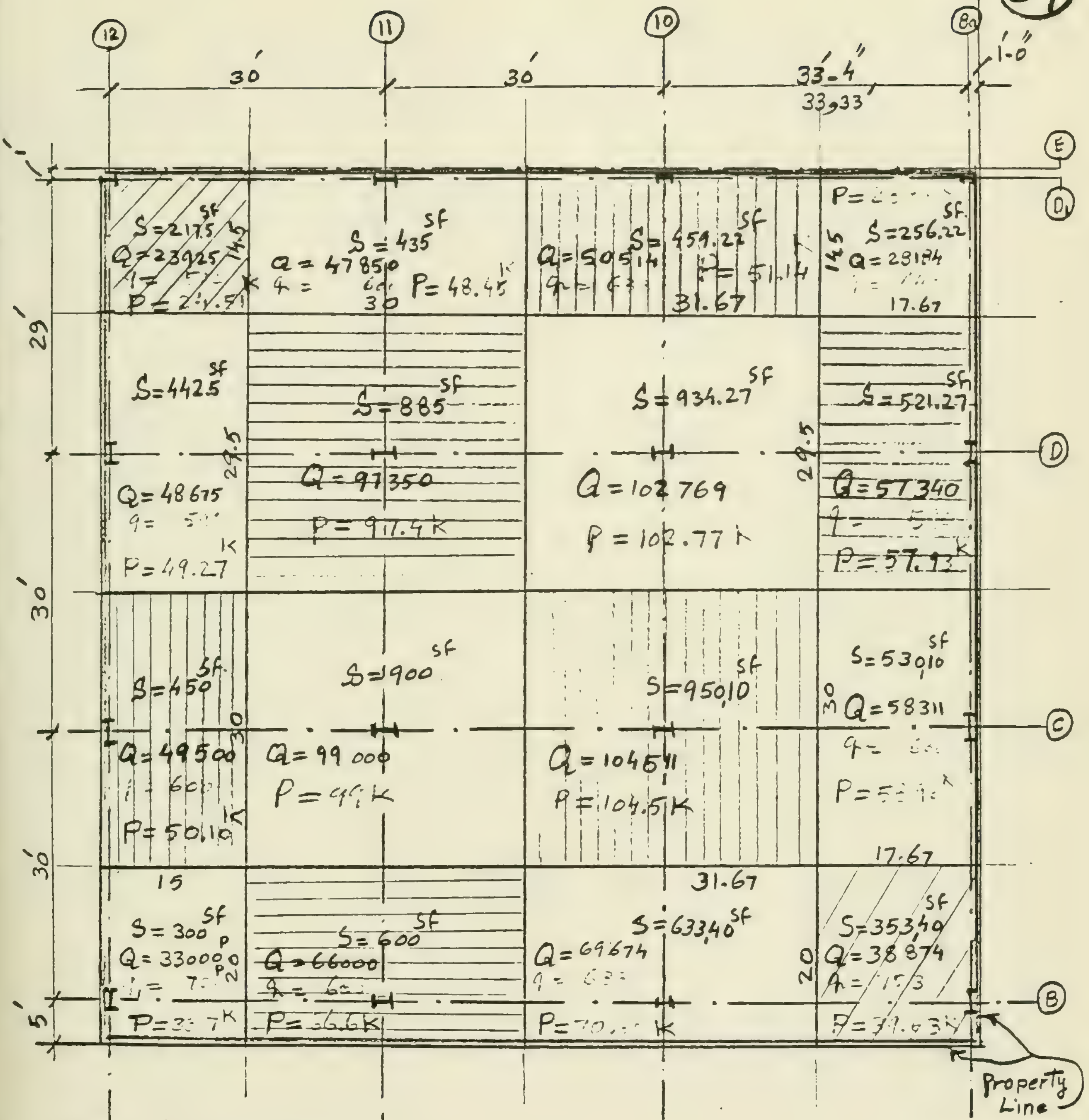


Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston Extension Part (A) Columns plan	DATE: April 20/1982 SCALE: 1" = 16'	DRAWING NO. Page (23) M.B.

38



Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston Extension Part (A) Columns plan (loads area)	DATE: April 20/1982 SCALE: 1" = 16'	DRAWING NO. 24 M.B.

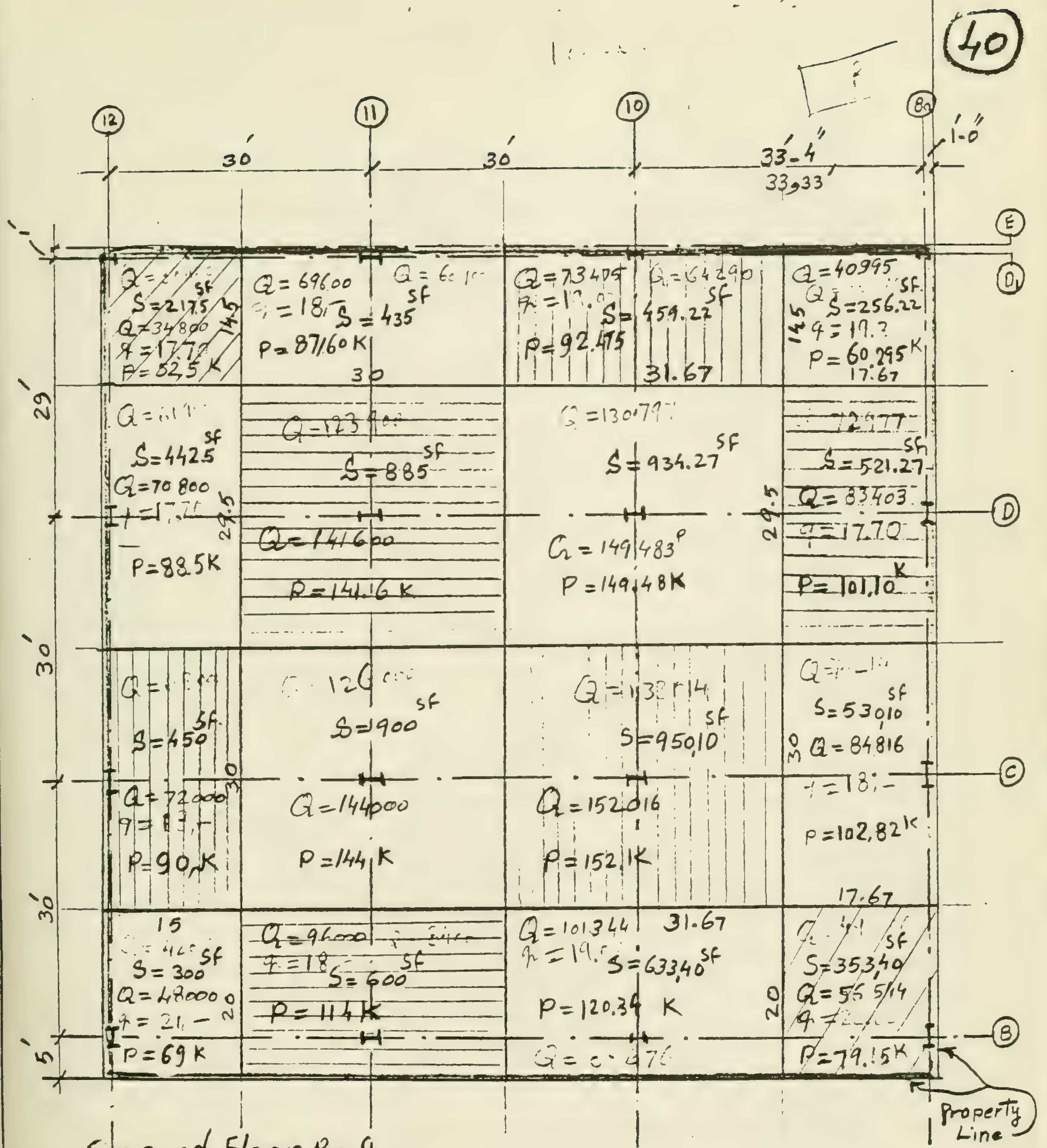


Third Floor Roof.

D+L = 110 P/sf

Parapet = 20 P/F

Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston Extension Part (A) Columns plan (loads area)	DATE: April 20/1982 SCALE: 1" = 16'	DRAWING NO. Page (25) M.B.



Second Floor Roof

$Q = D + L = 160$ P/sf $- 20$%

Side walls 30/sf of wall $\times 20'$ hight $= 600$ P/f

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Hynes Auditorium Boston, MA. Extension part (?)

41

Column's load on each floor

Mark	Parapet of Roof. Roof (3rd Floor)	walls and 2nd Floor Roof	walls and 1st Floor Roof	Ground Floor walls.
B-8a	753 lb, 38874 } 39.63 k	2260, 56544 } 79.15 k	79.15 k	22602 lb = 22.6 k
B-10	633, 69674 } 70.30 k	19,00, 101.33 } 120.34 k	120.35 k	19002 lb = 19. k
B-11	600, 66000 } 66.6 k	18.00, 96000 } 114 k	114. k	18000 lb = 18 k
B-12	700, 33000 } 33.7 k	21,–, 48000 } 69 k	69 k	21000 = 21 k
C-8a	600, 58311 } 58.9 k	18,–, 84816 } 102.82 k	102.81 k	18000 = 18 k
C-10	104.5 k	152. k	152. k	—
C-11	99 k	144. k	144. k	—
C-12	600, 49500 } 50.10 k	18.00, 72000 } 90. k	90 k	18000 = 18 k
D-8a	590, 57340 } 57.93 k	17.70, 83.403 } 101.10 k	101.10 k	17700 = 17.7 k
D-10	102.77 k	149.48 k	149.48 k	—
D-11	97.4 k	141.6 k	141.6 k	—
D-12	590, 48675 } 49.27 k	17.70, 70800 } 88.5 k	88.5 k	17.700 = 17.7 k
D_1-8a	643, 28184 } 28.83 k	19.30, 40995 } 60.295 k	60.3 k	—
D_1-10	633, 50514 } 51.14 k	19,00, 73475 } 92.475 k	92.48 k	—
D_1-11	600, 44850 } 48.45 k	18,00, 69600 } 87.60 k	87.60 k	—
D_1-12	590, 23925 } 24.51 k	17.70, 34800 } 52.5 k	52.5 k	—

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DATE Nov/5/8–
BY M.E.
NO. ~~28~~
OF

Hynes Auditorium, Boston, Ma. Extension part (2)

Column's load on each floor (after reduction)

42

Mark	Prapet of roof Roof (3rd Floor)	Walls, 2nd Floor roof	Walls. 1st Floor Roof	Ground Floor walls
B-8a	39.63 K	22.60 / 49.48 } 72.08 K		22.6
B-10	70.30 K	19,– / 88.68 } 107.69 K		19
B-11	66.6 K	18,– / 84,– } 102 K		18
B-12	33.7 K	21,– / 42,0 } 63,– K		21
C-8a	58.9 K	18,– / 74.21 } 92.21 K		
C-10	104.5 K	133.01 K		
C-11	99 K	126,– K		
C-12	50.10 K	18,– / 63,– } 81,– K		
D-8a	57.93 K	17.70 / 72.93 } 90.63 K		
D-10	102.77 K	130.80 K		
D-11	97.4 K	123.90 K		
D-12	49.27 K	17.70 / 61.95 } 79.65 K		
D_1-8a	28.83 K	19.30 / 35.87 } 55.17 K		
D_1-10	51.14 K	19,– / 64.29 } 83.29 K		
D_1-11	48.45 K	18,– / 60.90 } 78.90 K		
D_1-12	24.51 K	17.70 / 30.45 } 48.15 K		

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DATE Apr/26/5.
BY M.D.

NO. 27
OF

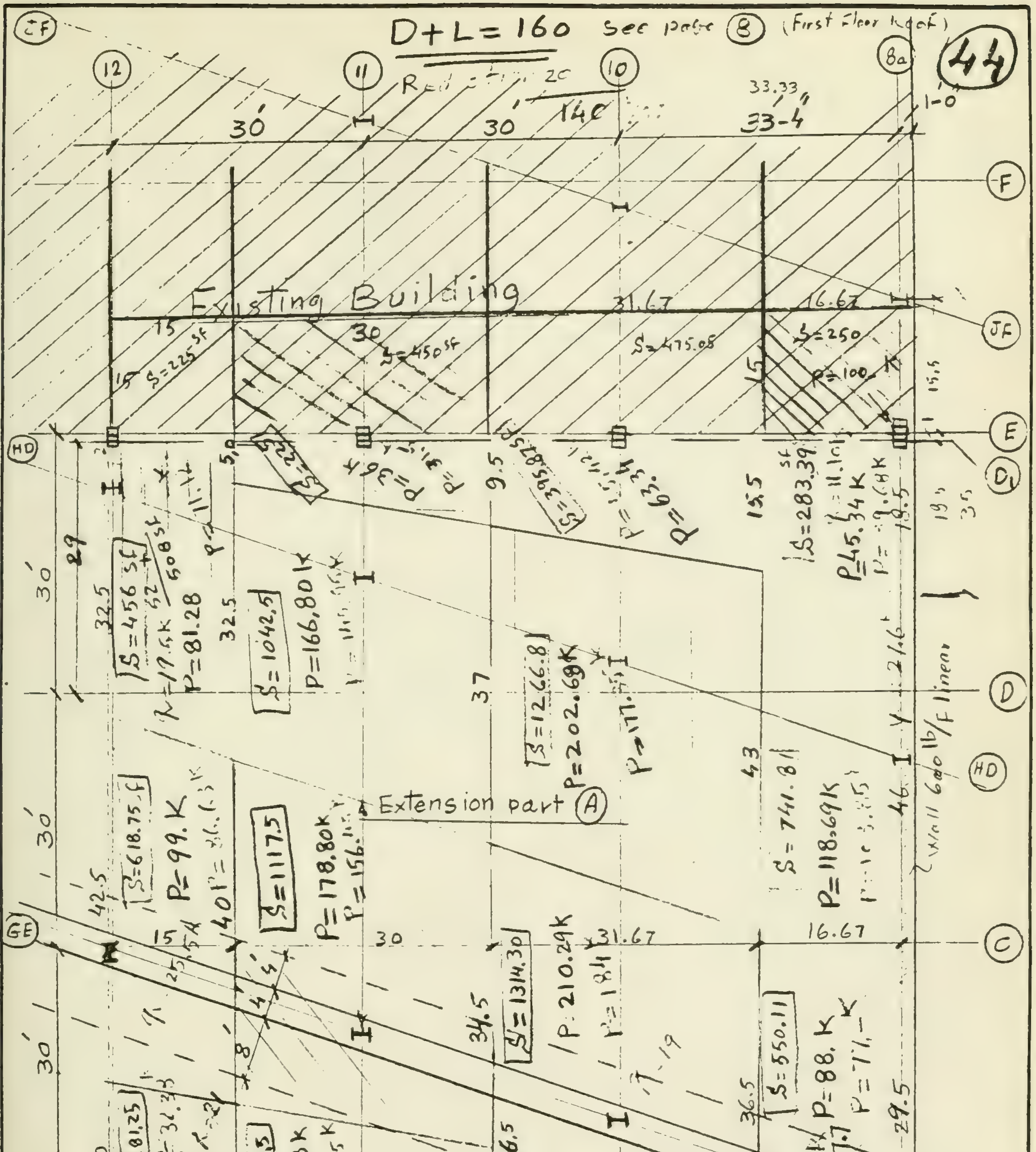
Hynes Auditorium Boston, M.I. Extension Part (A)

43

Columns Total load of colume on each floor.

after reduction

Mark	3 rd Floor	2 nd Floor	1st Floor	2nd Floor
B-8a	39.63 Kip	~~118.78~~ Kip	197.93 k	111.71 k
B-10	70.30 "	190.65 "	311.– k	177.7[illegible] k
B-11	66.6 "	180.60 "	294.6 k	16[illegible].60 k
B-12	33.7	102.7 "	171.7 k	96.7 -
C-8a	58.9	165.71 "	264.52 k	151.11 -
C-10	104.5	256.5 "	408.5 k	237.51 k
C-11	99	243 "	387.– k	225.– k
C-12	50.10	140.1 "	230.10 k	131.1 k
D-8a	57.93	159.03 "	318.06 k	14[illegible].61 k
D-10	102.77	252.57 "	402.37 k	233.57 k
D-11	97.4	239. "	390.6 k	221.3 k
D-12	49.27	137.77 "	226.27 k	123.92 k
D_1-8a	28.83	89.18 "	149.43 k	84.– k
D_1-10	51.14	143.62 "	236.1 k	134.[illegible] k
D_1-11	48.45	136.05 "	236.65 k	127.35 k
D_1-12	24.51	77.01 "	129.51 k	72.66 k



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NEW YORK, N.Y. CAMBRIDGE, MASS.

PROJECT: Hynes Auditorium Boston Extension part A (First Floor Roof) Load area & Load of 1st Floor Column	DATE: 26/8 SCALE: 1=K	DRAWING NO. Page 30 MB

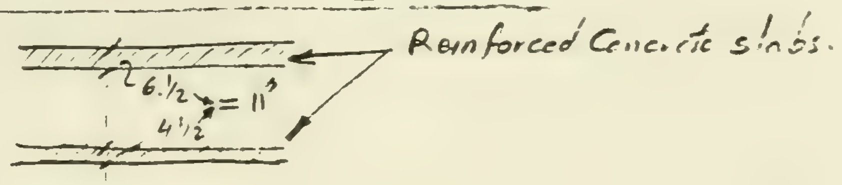
WEIDLINGER ASSOCIATES – CONSULTING ENGINEERS NEW YORK, N. Y. CAMBRIDGE, MASS.

DATE Apr. 26 BY F.B.

Hynes Auditorium, Boston, MA. Extension Part (F)

NO. 31 OF

Existing slab at Ground Floor (...Turnpike's Tunnel) (45)

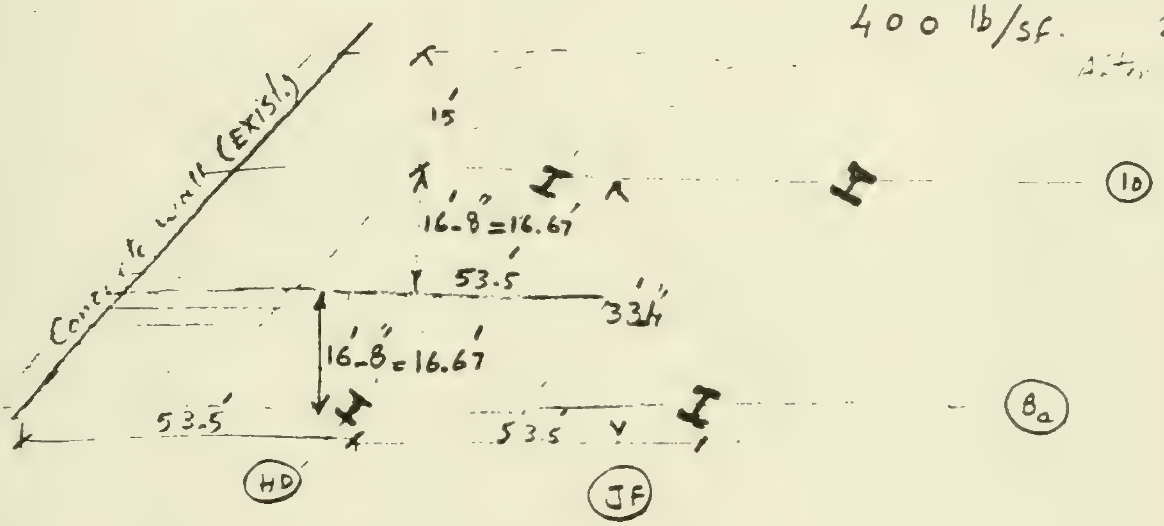


weight of slabs $= \frac{11 \times 145}{12} = 133$ lb/s.F.

live load 250 ← 100

beams and Girder and other 17

400 lb/sf. 250 R. After reduction



Col. {JF - 8a, HD - 8a} = 16.67' × 53.5' × ~~400~~ 250 = 356.74 k = 222.96 k

Col. {HD - 10, JF - 10} = (16.67' + 15') × 53.5 × ~~400~~ 250 = 677.34 k = 423.59

Col. {HD - 11, JF - 11} = 30 × 53.5 × ~~400~~ 250 = 642. k = 401.25

Col. HD - 12 = 2×15 × 53.5 × ~~400~~ 250 = 642 k. (Existing Roof of Turnpike) ~ 401.25

46

Total Load on Ground Floor Columns. @ Frame 8a

① GE-8a Col.

			After Re...
	Ground floor Roof slab (page 30)	88,- kip	77
page 30	From second floor wall (29.5 + 16.67) x 600 =	27.70	27.70
	" uper column (B-8a) page 29	118.87	111.71
	a part from column (G-8a) $\frac{\cancel{161.71}\ 151.11 \times 22}{55}$ = (See page 29 / page 41)	64.68	[illegible]
	Total load of Col. GE-8a	299.25 K	276.35 k

② HD-8a Col.

	Ground floor Roofslab page 30	118.69 K	10[illegible]
page 30	From second floor wall (46 x 600) =	27.60	27.60
page 29	a part from second floor col. G-8a = $\frac{\cancel{161.71}\ 151.11 \times 33}{55}$ =	97.03	90.66
page 41	" " " " " D-8a = $\frac{\cancel{159.03}\ 143.[illegible] \times 29}{37}$ =	124.64	116.43
	Total load of Col. HD-8a	367.96 K	335.59 k

③ D_1-8a Col.

	Ground floor Roof slab (page 30)	45.34 K	39.65
page 30	From Second floor wall (18.5 x 600) =	11.10 K	11.10
page 29	a part of second floor col. D-8a ($\cancel{159.03}$ 143.[illegible] – 124.64 116.[illegible])	34.39 k	32.13
	From uper Col. D_1-8a	89.18	84.–
	Total load of Col. D_1-8a	180.01 k	166.[illegible]

Total Load on Groud floor Columns. (b) Frame (10)

			[illegible]
(1) GE-10 Ground floor Roof slab page 30	210.29	k.P.	184.–
Page 30 From Second floor wall (31.67 × 600) =	19.00	"	19.–
" uper Col. B-10 (page 29, 2nd floor)	190.65	"	177.[illegible]
(Page 29) a part " " " C-10 ($\frac{\cancel{256.5}^{237.51} \times 30}{55}$) = (See page 42)	139.91	"	[illegible].55
(Page 42) " " " " " D-10 ($\frac{\cancel{252.57}^{233.57} \times 4}{55}$) = "	18.37	"	16.99
Total Load of Col. GE-10	578.22	K	527.52 K

(2) HD-10

Ground floor Roof slab (page 30)	202.69	k.P.	177.[illegible]
(Page 29) a part From uper Col. C-10 ($\cancel{256.5} - \cancel{139.91}$ 237.51 − 129.55) =	116.59	"	[illegible]
" " " " " " D-10 (252.57 − 18.37) = 233.57 − 16.99	234.20		216.58
Total Load of Col. HD-10	553.48		501.[illegible] k

(3) D_1-10

Groud Floor Roof slab (page 30)	63.34	k.P.	55.42
From uper Col. (page 29 2nd floor)	143.62	"	[illegible]
Total Load of Col HD-10	206.96	k.P	[illegible],35 K

48

Total Load on Ground floor Columns. (C) Frame (11)

				After reduction
(1) B-11				
	Ground Floor Roof slab (Page 30)	70.8	Kip.	61.95
(Page 30)	From second floor wall (30 x 600)	18,00	"	18.00
(Page 29)	" uper Col. B-11 (2nd floor)	180.60	"	[illegible]
	Total Load on Col. B-11	269.40		243.55
(2) GE-11				
	Ground Floor Roof slab (Page 30)	178.80	Kip	156.45
(Page 29) (Page 43)	a part from uper Col. C-11 $\left(\frac{\overset{225}{\cancel{243}} \times 44}{55}\right)$ (See page 43)	194.40	"	130,-
"	" " " " " D-11 $\left(\frac{\overset{221.3}{\cancel{239}} \times 14}{55}\right)$ " " "	60.84	"	56.33
	Total Load on Col. GE-11	434.04	k	392.70
(3) HD-11				
	Ground Floor Roof Slab (Page 30)	166.80	k	145.95
	a part from uper col. C-11 ($\overset{225}{\cancel{245}} - \overset{180}{\cancel{194.40}}$)	48.60	"	45,-
	" " " " " D-11 ($\overset{221.3}{\cancel{239}} - \overset{56.33}{\cancel{60.84}}$)	178.16	"	164.17
	Total Load on Col. HD-11	393.56	k	355,12
(4) D_1-11				
	Ground floor Roof slab (Page 30)	36,-	k	31.5
	From uper Col. D_1-11 (page 29, 2nd Floor)	136.05		127.[illegible]
	Total Load on Col. D_1-11	172.05		15[illegible].95

49

Total Load on Ground Floor columns (d) Frame (12)

				After Reduction
(1) B–12 Col.				
	Ground Floor Roof slab (page 30)	45.–	Kips	32.3[illegible]
(page 30)	from second floor wall (15+5+15)x600 =	21,–	"	21,–
	" uper Column (B–12) (page 29, 2nd floor)	102.70	"	96.[illegible]
	Total load of Col. B–12	168.70	Kips	150,09 K
(2) GE–12 Col. or C–12				
	Ground Floor Roof slab (page 30)	99,–	kip	[illegible]
(page 30)	From second floor wall (42.5 x 600)	25.50	"	25.50
	From uper Column (GE or C–12) (page 29, 2nd floor)	140.10	"	131.1[illegible]
(page 29) (page 44)	a part from second floor Col. (D–12) $\frac{\cancel{13277}\ 123.92 \times 25}{55}$ = (page 44)	62.62	"	53.6[illegible]
	Total Load of Col. GE–12	327.22	Kip	301.3[illegible]
(3) HD–12 Col.				
	Ground floor Roof slab (page 31)	81.28	kip	71.12
(page 31)	From second floor wall 32.5x600 =	19.50	"	19.50
	From uper Col. D_1–12 (Page 29, 2nd floor)	77.01	"	72.[illegible]
(page 29)	a part " " " (D–12) $\cancel{13277 - 62.62}$ 123.92 – 53.6[illegible] =	75.15	"	70.3[illegible]
	Total Load of Col. GE–12	252.94	Kips	233.6[illegible]

WEIDLINGER ASSOCIATES – CONSULTING ENGINEERS NEW YORK, N. Y. CAMBRIDGE, MASS.

DATE 5/12/82
BY M.B

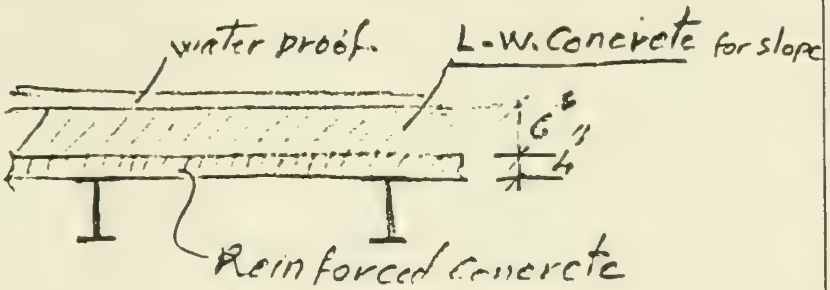
Hynes Auditorium, Boston, MA. Extension

NO. 8
OF 36

50

Roofing.

(Existing)



D.L

Conc. slab	50
sloping	35
water proofing	5
	90 lb/sf

L-L

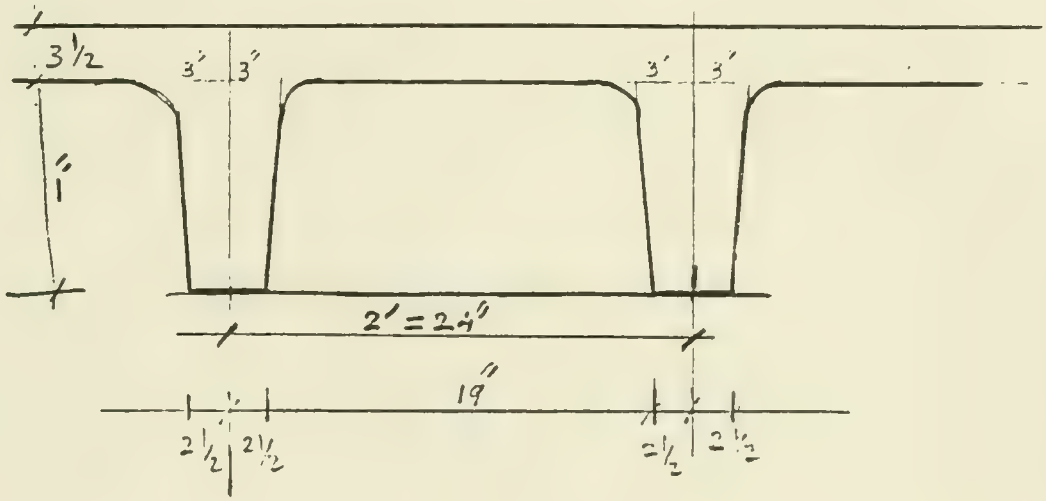
snow	30
Fall ceiling and Mechani	15
	45

Total Load per square foot 135 lb/sf

Tynes Auditorium, Boston, MA. Extension

51

Second floor slab. (Existing)



weight of Two ways Ribd slab. for a part $2' \times 2' = 4^{sf.}$

$$\text{slab} = 2' \times 2' \times 0.2917' \times 150^{lb} = 175.02$$

$$\text{Ribs} = \left[\left(4 \times \frac{2.5'' + 3''}{2} \times (19'' + 2.5'') \times 1'\right) \div 144\right] 150 = 246.35$$

$$421.35$$

$$\frac{421.35}{4} = 105.30 \text{ lb/sF.}$$

add 10% for bean $105.30 \times 1.10 = 116$ lb/sF.

D.L = slab 116
Finishing floor 15
Other 9
140 lb/sF.

L.L = Exhibition area 250 lb/sf
Ceiling and Mechanic 20
270

Total Load 140+270 = 410 lb/sF.

WEIDLINGER ASSOCIATES – CONSULTING ENGINEERS NEW YORK, N. Y. CAMBRIDGE, MASS

DATE April/27/[illegible] BY M.A.

Hynes Auditorium Boston, [illegible] Extension part (A)

NO. 3 OF

52

Turnpike Columns. load

HD – 8a :

			After [illegible]
(Page 31)	From Ground Floor slab (Turnpike Roof)	356.74 k	222.[illegible]
	from uper Column (Page 32)	367 "	[illegible].59
(Page 30)	weight of wall of Ground Floor $46 \times 600 =$	27.60 "	27.60
(Page 32) (Page 41)	a part of Load of D_1-8a Col. $\left(\frac{\cancel{180.01} \times (\cancel{15}.5+1)}{53.5}\right)$ 166.31 × 16	55.52 "	51.47
(Page 30)	a part of weit of Wall neer $D_1-8_a \left(35-\frac{53.5}{2}\right)600 =$	4.95 "	4.95
(Page 41)	a part of load at (D-8a) Existing building $\frac{145 \times 15.5}{53.5} =$	42.– "	42.–
	Total Load of Col. HD-8a	853.86 K	687.57 K

Turnpike Columns Load

HD–10

		After Reduction
From Ground Floor slab (page 31)	677 k	423.59
" uper Column (page 33)	553.48 k	501.39
page 33 } page 42 } apart of Column D,–10 $\frac{\cancel{206.96}^{184.35} \times 28.5}{53.5} =$	110.25 k	100.94
(Page 42) " " " " D–10 $\frac{258.87 \times 285}{53.5} =$ (Existing building)	137.90	137.90
Total load of Col. HD–10	1482.72 k	1164.52

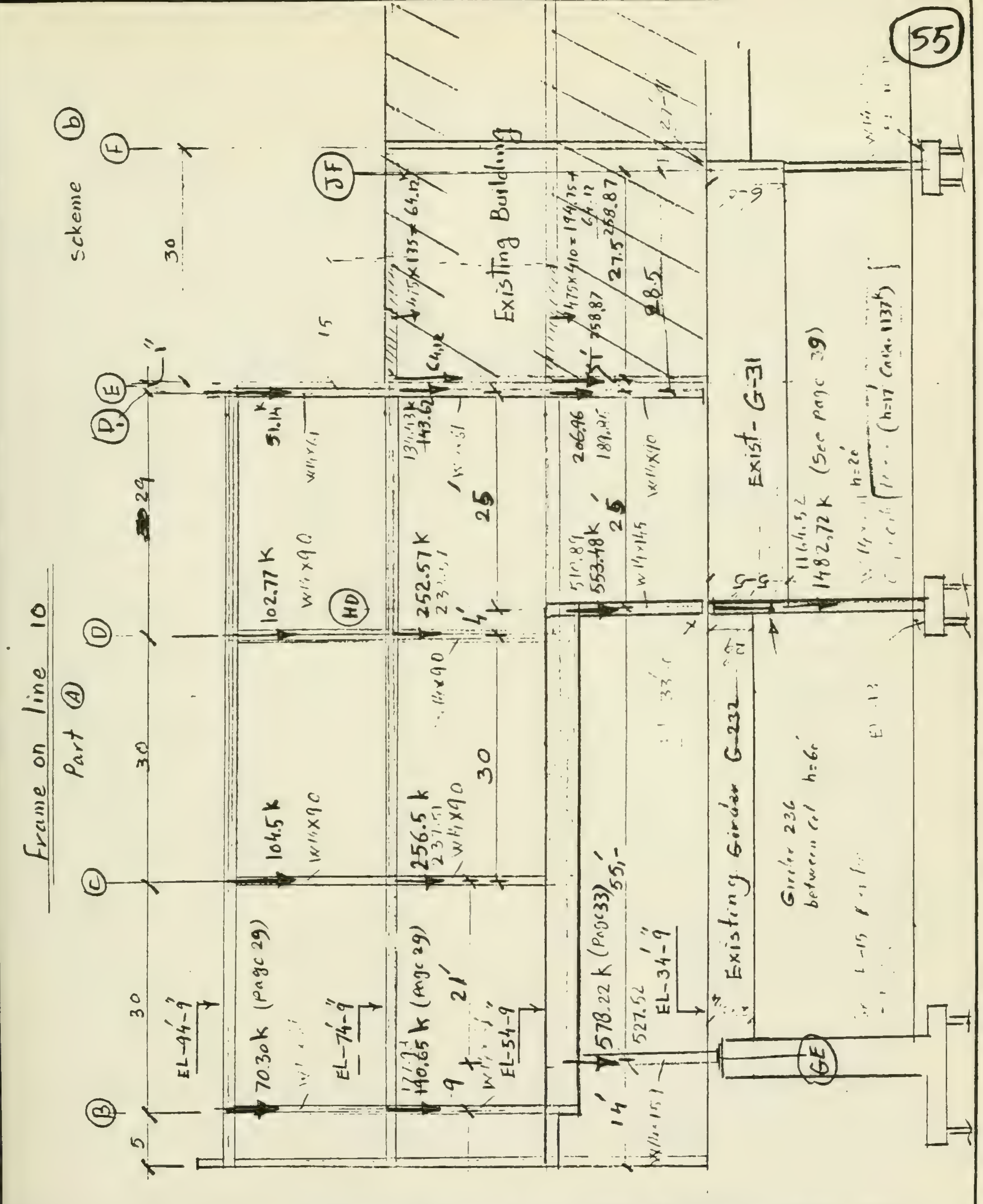
HD – 11

From Ground Floor slab (page 31)	642 k	451.25 k
From uper Column (page 34)	393.56	355.92
(page 34) } (page 43) } a parat of Column D,–11 $\frac{\cancel{172.05}^{158.35} \times 38.5}{53.5} =$	123.81	114.31
(Page 43) a parat of column D–11 (Exist. Build.) $\frac{245.25 \times 37.5}{53} =$	171.90	171.90
(Page 43) a part of Column F–11 " " $\frac{490.5 \times 7.5}{53.5} =$	68.76	63.76
Total load of Col HD–11	1400.03 k	1112.14 k

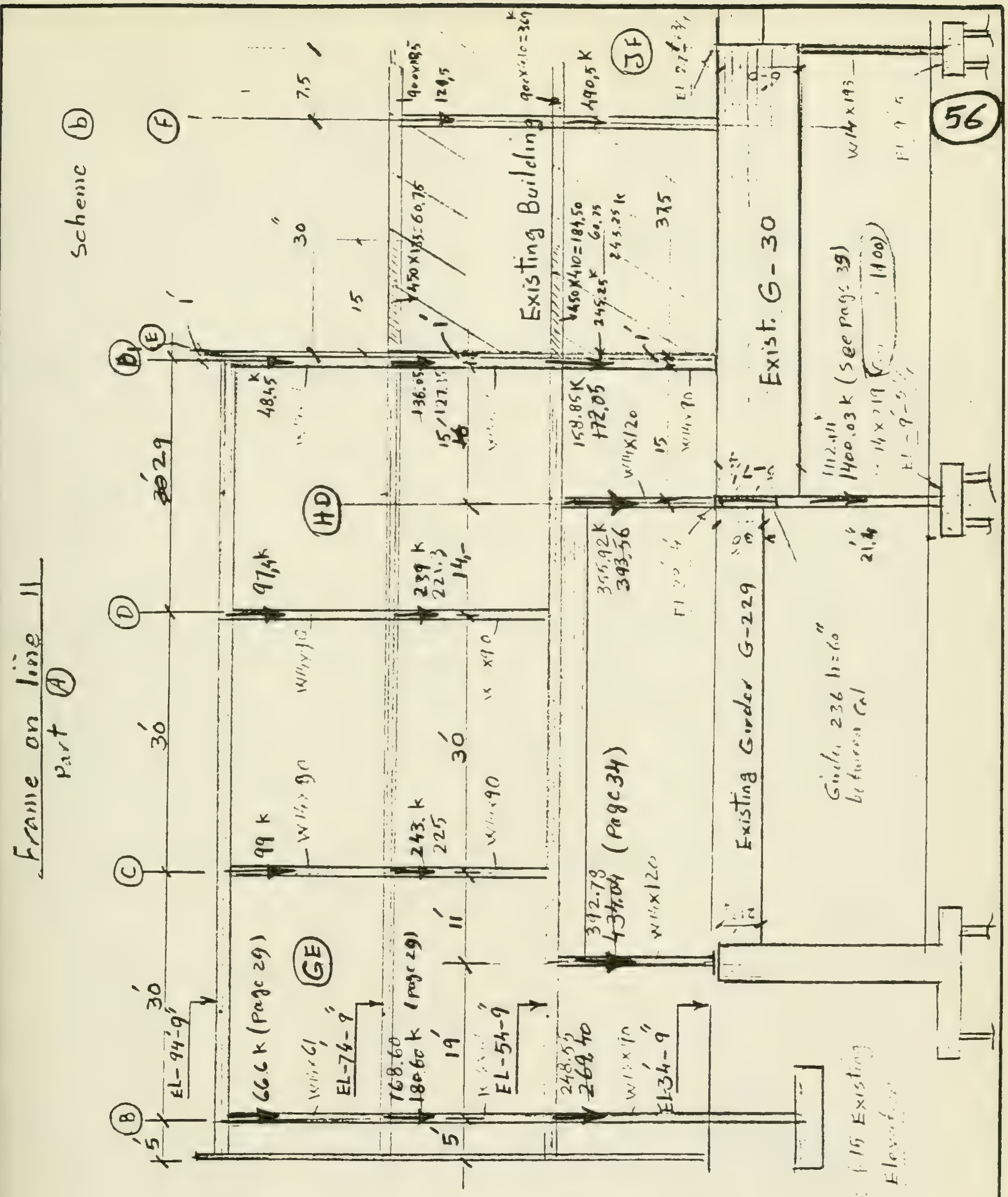
Turnpike Column load

HD-12

		After Reduction
From Ground Floor slab (page 31)	642. K	401.25
From uper Column (page 35)	252.94 "	233.6[illegible] K
(page 30) weight of wall of Ground floor 53.5 x 600	32.10 "	32.10
(Page 44) a part of load of Col. E-12 (Exist. Build.) $\frac{132.55 \times 49.5}{53.5}$ =	122.64 "	122.64
" " " " " " " " F-12 " " $\frac{265.10 \times 18.5}{53.5}$ =	91.67 "	91.67
Total load of Col. HD-12	1141.35 K	881.[illegible] K



Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston Extension Part (A)	DATE: April 26/82 SCALE: 1" = 16'	DRAWING NO. Page 42 M.B



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CONSULTING ENGINEERS
NEW YORK, N.Y. CAMBRIDGE, MASS.

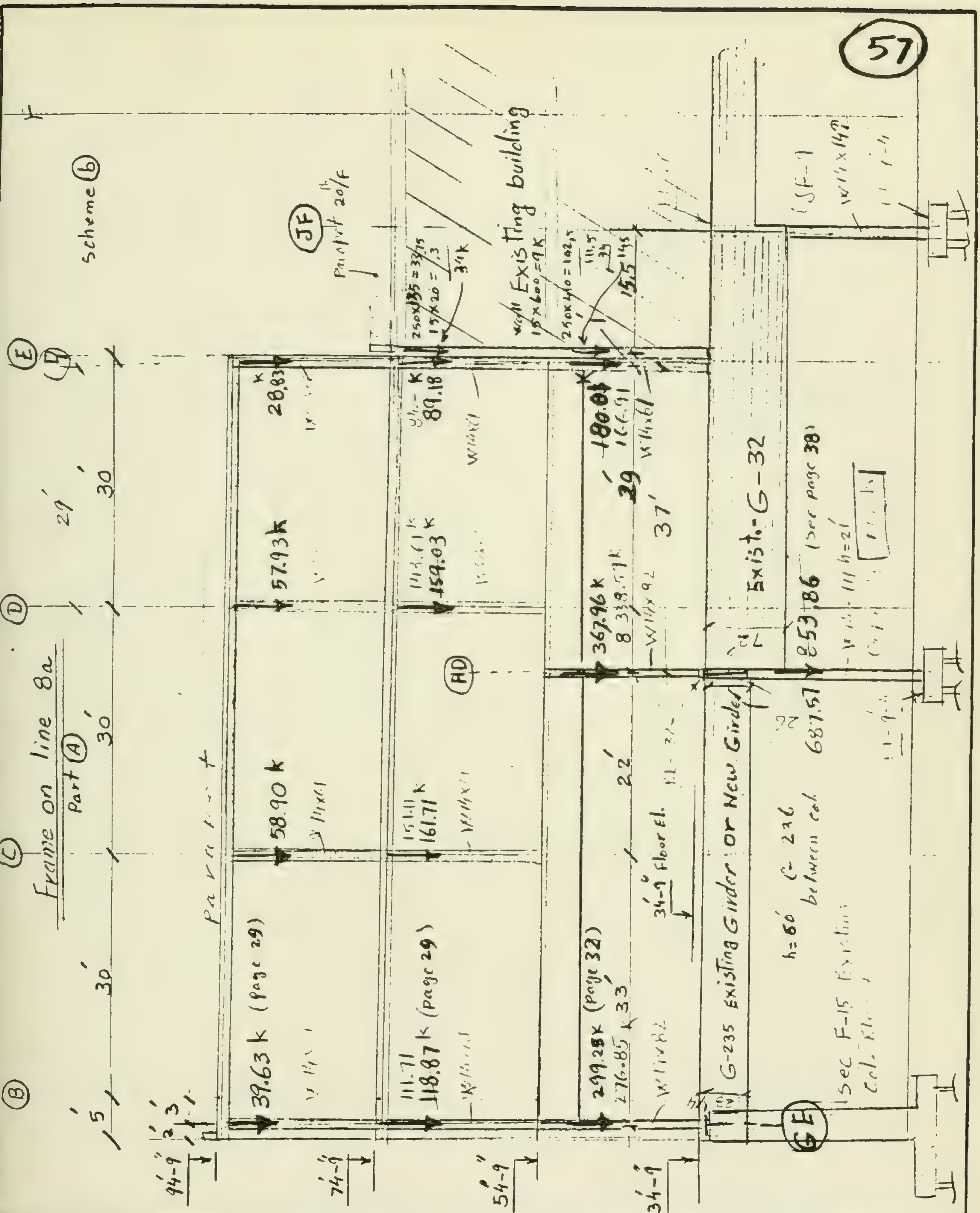
PROJECT: Hynes Auditorium Boston
Extension Part (A)

DATE: April 26/82

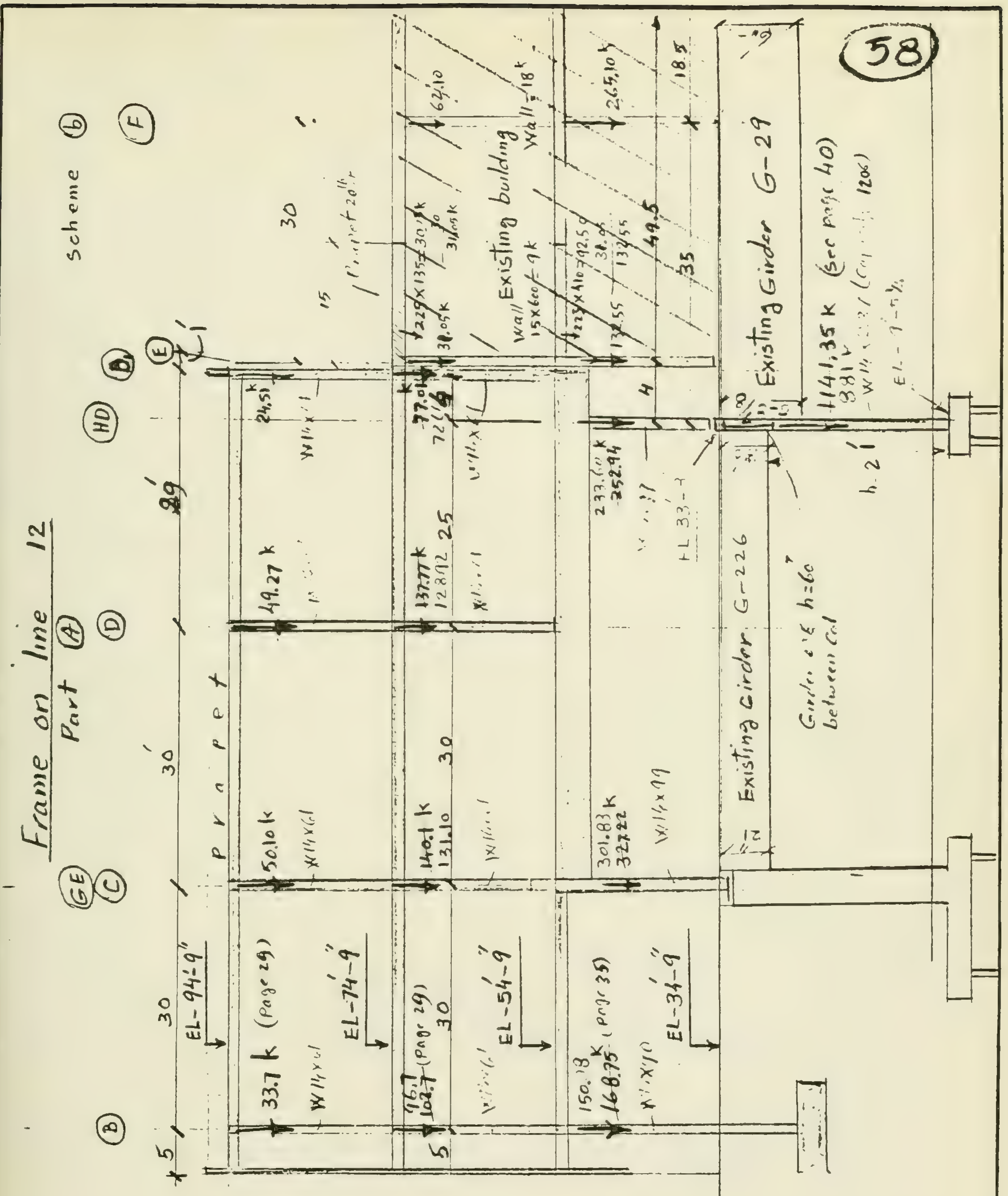
SCALE: 1" = 16'

DRAWING NO. Page 43
M.B

57



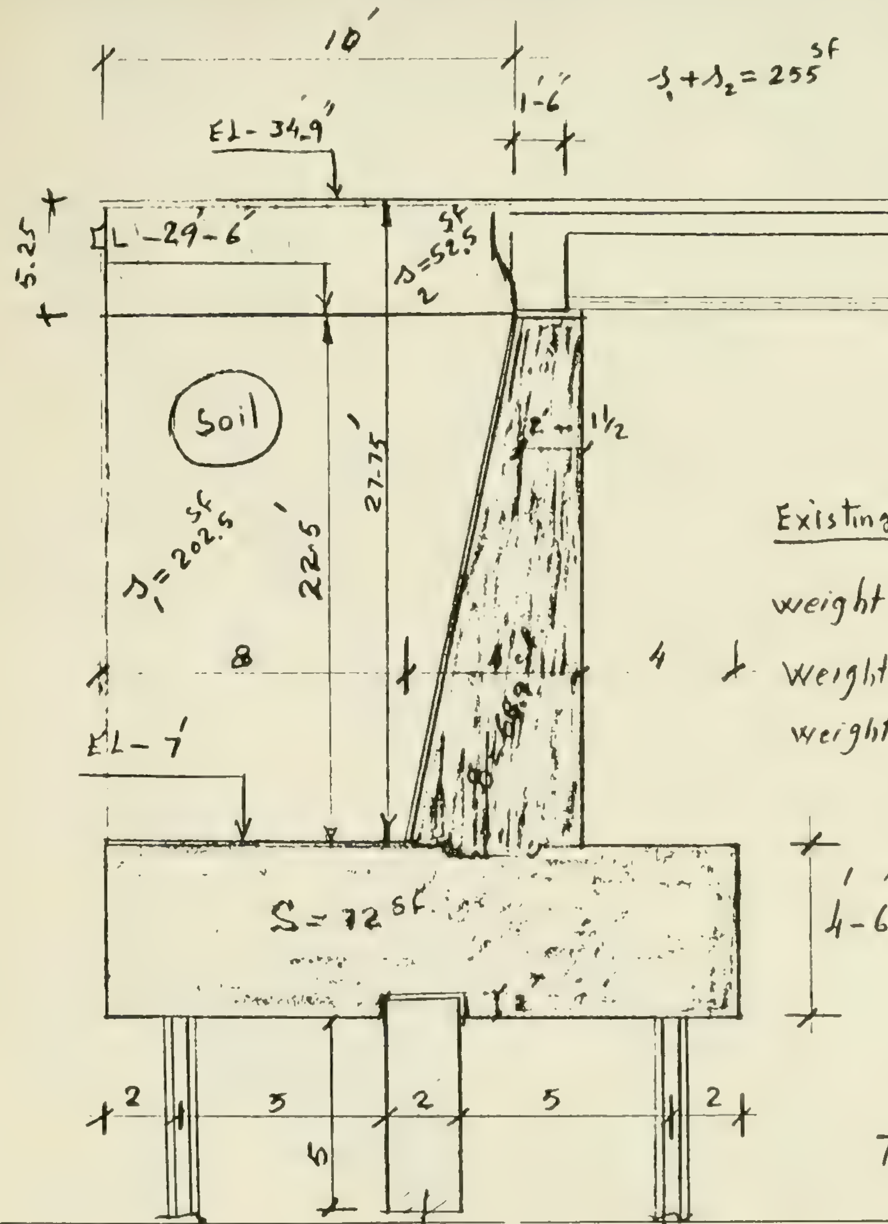
Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Extension (part (A))	DATE: April 26/82 SCALE: 1" = 16'	DRAWING NO. Page 41 M.B.

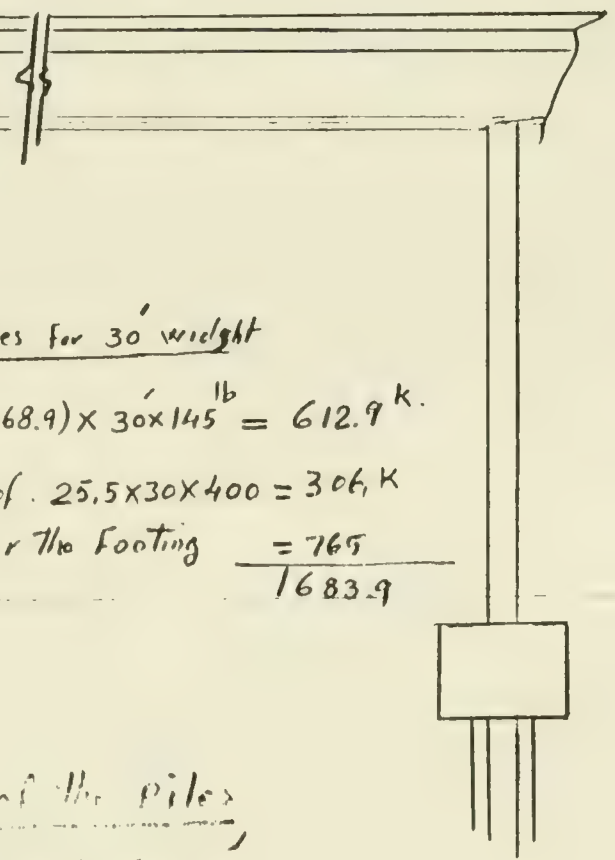


Weidlinger Associates CONSULTING ENGINEERS NEW YORK, N.Y. CAMBRIDGE, MASS.	PROJECT: Hynes Auditorium Boston Extension part (A)	DATE: April 26/82 SCALE: 1" = 16'	DRAWING NO. Page 44 M.B

Assume for Dencity of soil = 100 lb/cuf

$s_1 + s_2 = 255^{sf}$ 255 × 30 × 100 = 765 K





Existing Load on the piles for 30' widght

weight of wall. $(72+68.9) \times 30' \times 145^{lb} = 612.9^{k}$.

Weight of half of roof. $25.5 \times 30 \times 400 = 306\ K$

weight of Soil over the Footing $= 765$

1683.9

Capacity of the piles Under the wall for 30'

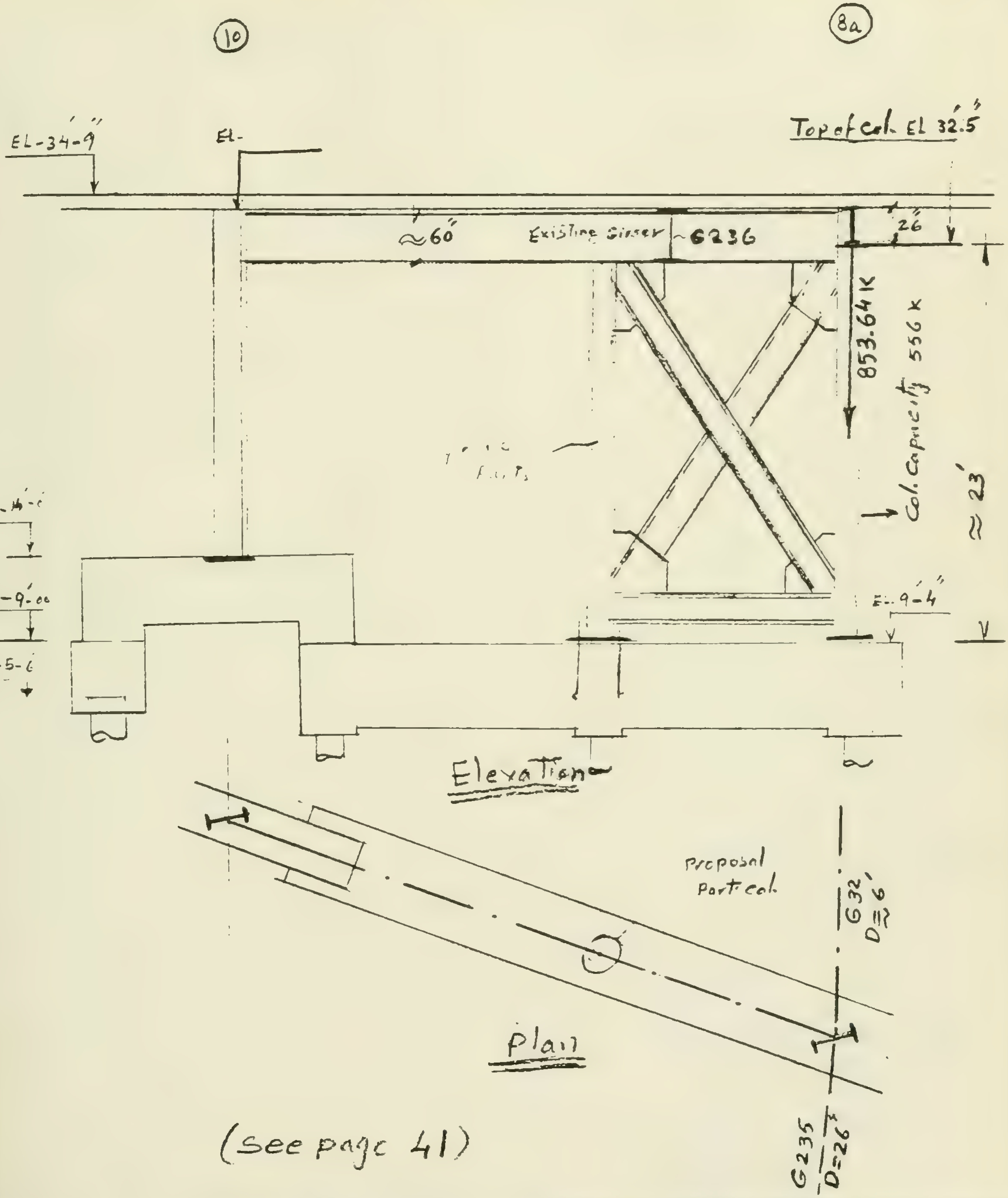
Ten piles: Each $120^{Ton} = 240^{kip} \times 10 = 2400^{kip}$.

Allowable capacity $= 2400 - 1683.9 = 716.10\ K$.

$2 \times 5 = 10 \times 145 = 1450\ ^{lb}/_{2sf}$.

σ_{soil} = at least $\frac{1450}{2 \times 144} = 5\ ^{lb}/_{s.in}$

Existing Piles & Columns on line HD



(see page 41)

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PROJECT: Hynes Auditorium, Boston, Extension part ②
Column HD-8a on Turnpike Level

DATE: May 3/1982

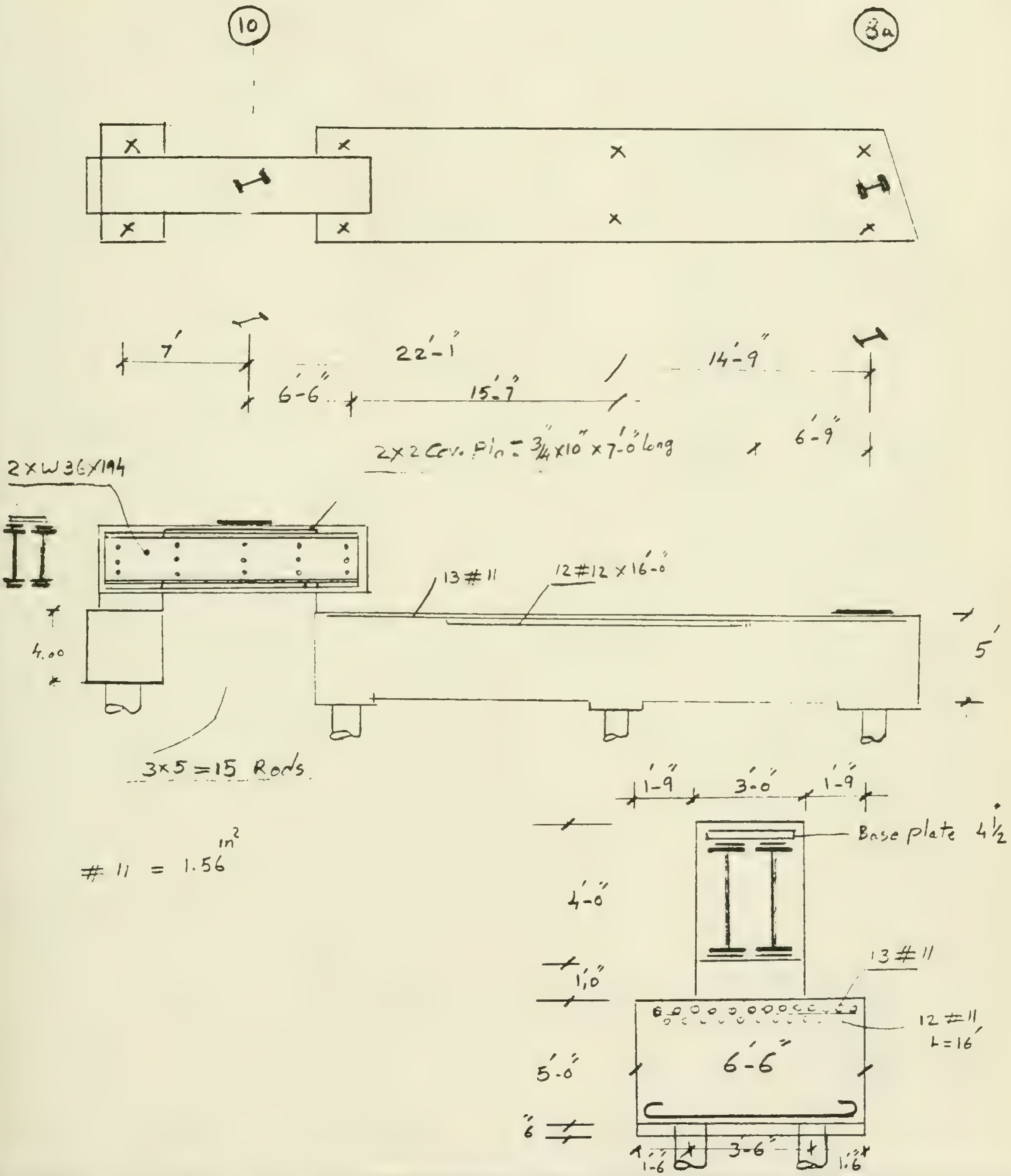
SCALE:

DRAWING NO. Page 47

Piles & columns on line HD

Existing situation

61



$\# 11 = 1.56\ in^2$

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NEW YORK, N.Y. CAMBRIDGE, MASS.

PROJECT: Hynes Auditorium, Boston, Extension part (A)

Column (HD-8a) on Turnpike Level

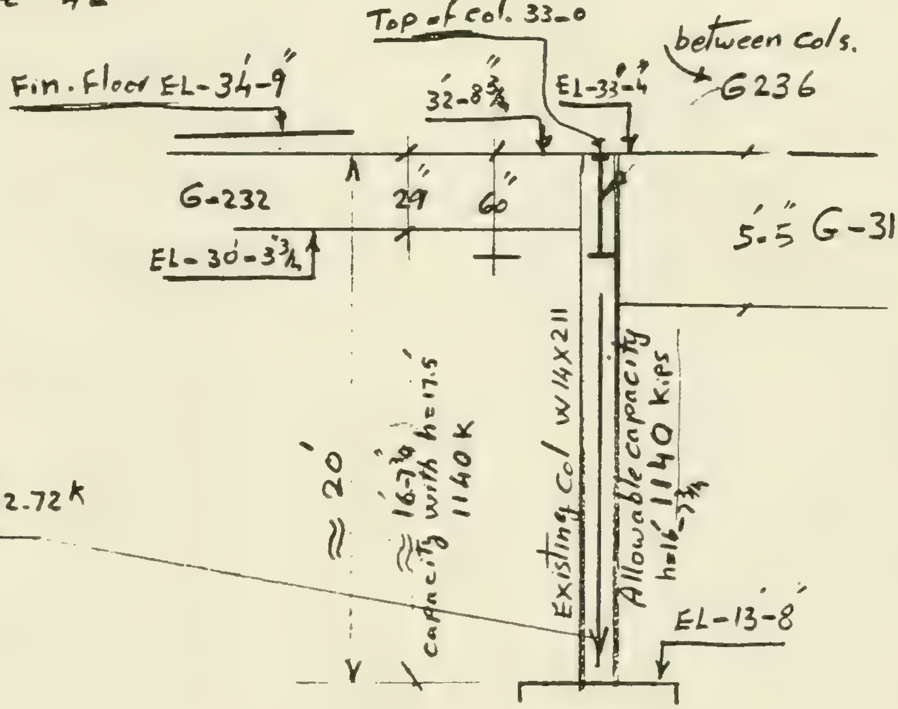
DATE: May 3/1982

SCALE:

DRAWING NO. page 46

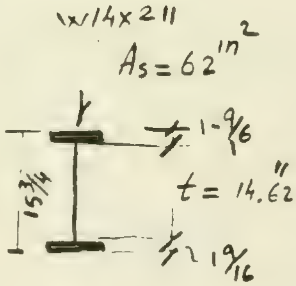
Column HD-10 on Turnpike Level

see page 42



Total Load 1482.72^k

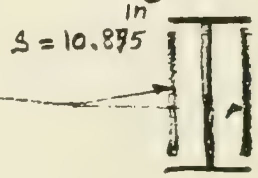
$1482.72 - 1140 = 343$ kip



Existing stress capacity $= \frac{1140}{62} = 18.38\ ^k/in^2$

$343 \div 18.38 = 18.66^{in^2} \rightarrow 20^{in^2} \rightarrow 2 \times 10^{in^2}$

use plate 3/4" tick 14 1/2



Load Capacity of Col.

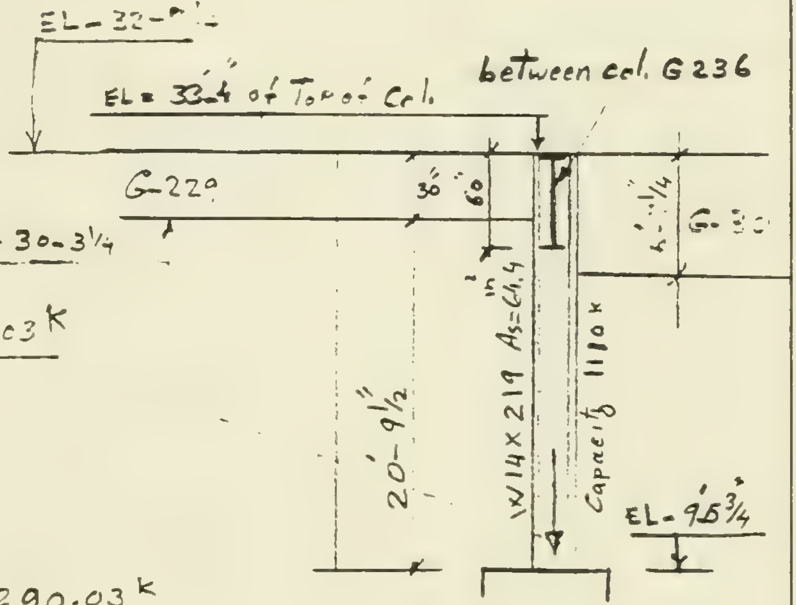
$62 + 2 \times 10.875 = 83.75^{in^2} \times 18.38 = 1539.39 > 1482.72^k$

In addition Allowable stress of box Col. is more than 18.38 ok

(63)

Col. HD-11 on Turnpike Level

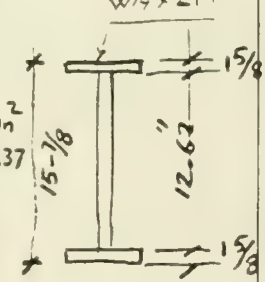
See page 43



Total Load 1400.03^k

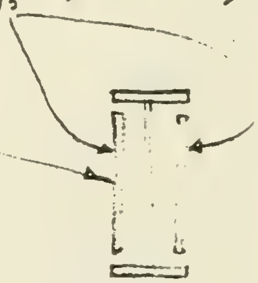
$1400.03 - 1110 = 290.03^k$

Existing stress capacity $= \dfrac{1110}{64.4} = 17.23\ ^k/in^2$



$290.03 \div 17.23 = 16.83\ in^2 \rightarrow 2 \times 9 \rightarrow 2 \times 9.37\ in^2$

use plate ¾" tick 12½" widght $S = 9.375\ in^2$



Load capacity of col.

$64.4 + 2 \times 9.375 = 83.15\ in^2$

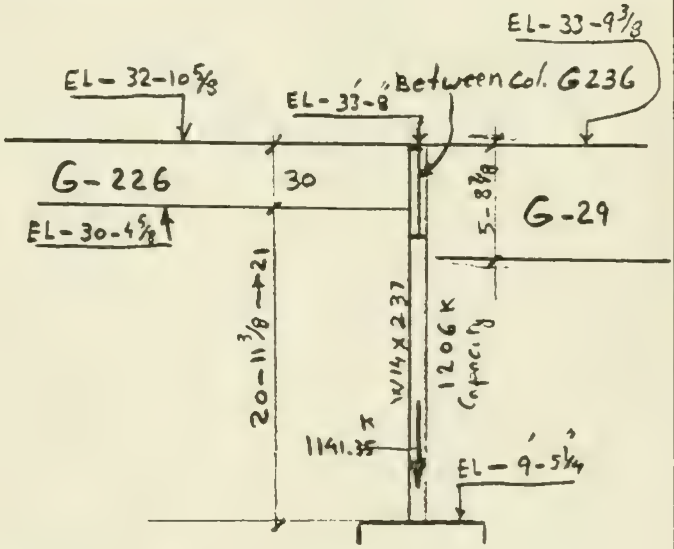
$83.15 \times 17.23 = 1432.67^k > 1400.03^k$ ok

In addition Allowable stress in box Col. is more than $17.85\ ^k/in^2$

ok

Col. HD-12 on Turnpike Level.

See page 44



Capacity = 1206 > 1141.35^k ok

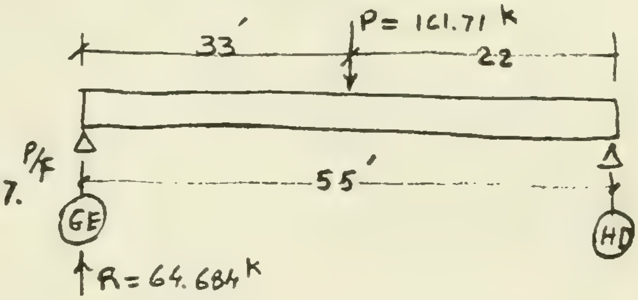
Hynes Auditorium, Boston, Extension part (2)

(65)

Plate Girder of Ground Floor Roof on line 8a

$P = 161.71$ See page 41

(see page 31)
$w = 16.67 \times 160 + 600$ wall $= 3267.$ $^{P}/_{F}$



$M_p = 64.684 \times 27.5 = 1778.81$ F.k.

$M_w = \dfrac{3.267 \times 55 \times 55}{8} = 1235.33$ "

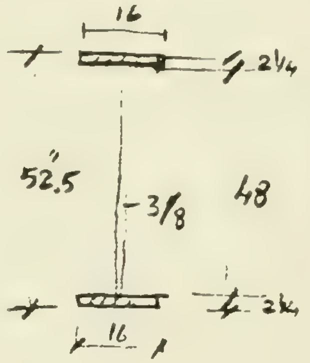
$M = M_p + M_w = 3014.14$ F.k.

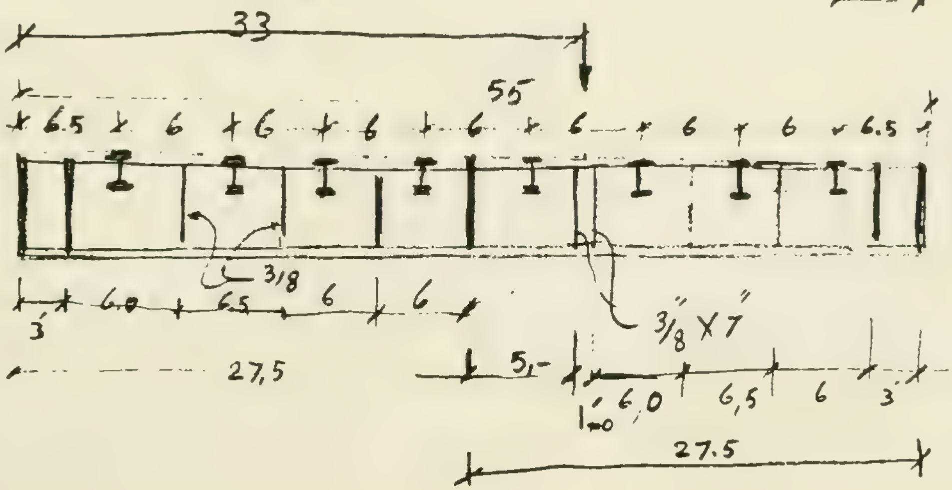
Required section modulus. $S = \dfrac{M}{\sigma} = \dfrac{3014.14 \times 12}{22} = 1644.07$

Assume p.G as shown.

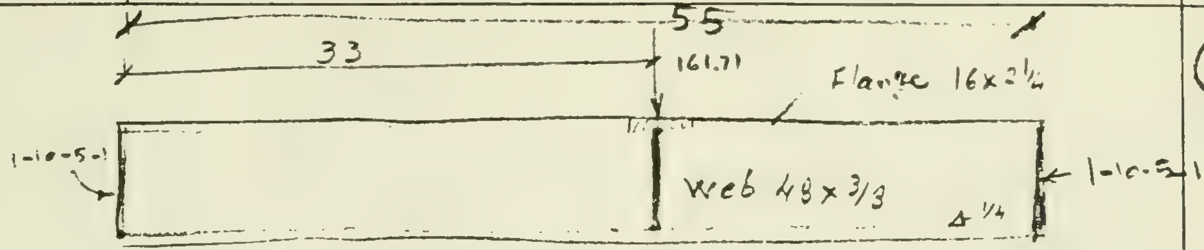
$S = 1860$ in^3 $A_s = 90$

wt per foot = 306





Hynes Auditorium, Boston, M.A. Extension Part (2)



Check Web crippling (AISC) See page 5-35 $\frac{R}{t(N+2K)} \leq 0.75F_y = 27^k$ Formula (1-10-8)

(1) under concentrated load at 33'

$P = R = 161.71\ k$ $t_w = 3/8 = 0.375$ $N = 10^{in}$ $k = 2\frac{1}{4} + \frac{1}{4} = 2.5$

$\frac{161.71}{0.375(10+2\times2.5)} = \frac{161.71}{5.625} = 28.75 > 27$ stifner is required

(2) web of plate girder $h/t = \frac{48}{0.375} = 128$ $a/h = \frac{72}{48} = 1.5$

as Table 10-36 page 5-20 Allowable shear stress = 6.6 k.

$or = \left[5.5 + \frac{4}{(a/h)^2}\right] \frac{10{,}000}{(h/t)^2} = 4.44\ kip$ Formula (1-10-10)

Structural Calculations

Commercial Block C

Weidlinger Associates

(67)

Loads:

(a) Roof. use 1½ Blok Deck

weight of Concrete and deck	36	lb/sf
" " Beams	10	
" " ceiling and mechanical	15	
" " water proofing and Gravel	15	
" " Snow	30	
Total weight of roofing	106	P/sf

$\boxed{\text{Assume } 110\ P/sf}$

(b) Typical floor

weight of slab as above	36	P/sf.
" " Beams	10	"
" " ceiling + Mechanical	15	"
" " L.L.	100	"
Total weight of Typical Floor slab	161	P/sf

$\boxed{\text{Assume } 160\ P/sf}$

(c) If 250 P/sf L.L is considered

$160 + 150 = 310\ P/sf$

$\boxed{\text{Assume } 325\ P/sf}$

(68)

weight of Existing Commercial floor on strees level

① D.L

weight of: Concrete slab $\frac{150\times5.5}{12}$ = 68.75 P/SF.

" Flooring 4" Concrete 50.00 "

" Beame 11.25 "

Total. D.L. 130.00 P/SF

② L.L 250.00 "

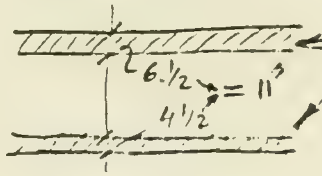
Total load 380.00 P/SF

weight of:- Turnpike Roof.

H~~ynes Auditorium~~ Commercial Center, Boston, MA. Extension part (A)

Existing slab at Ground Floor (Roof of Turnpike's Tunnel)

(69)

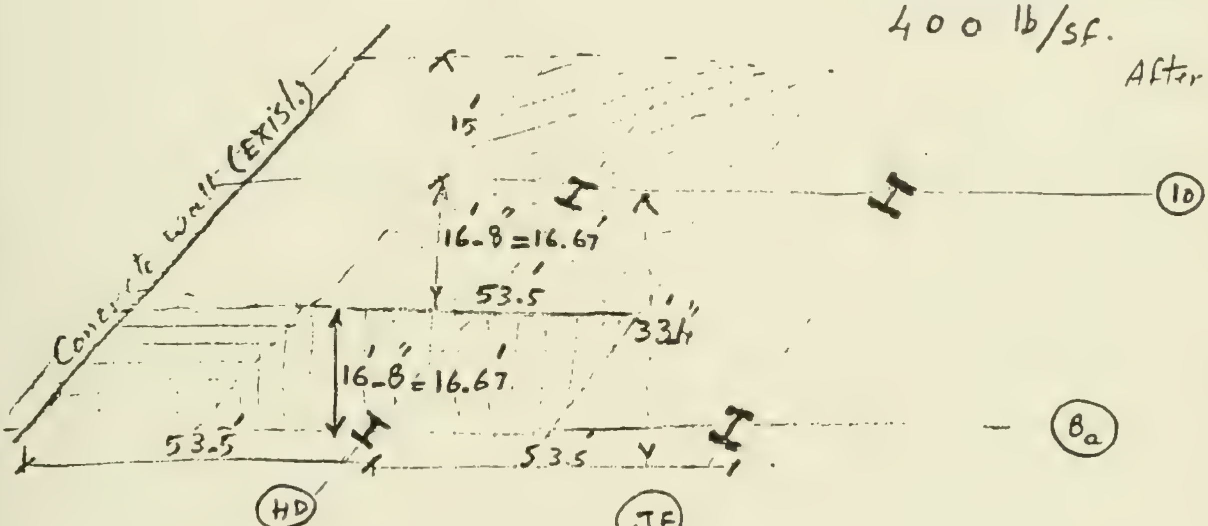


weight of slabs $= \frac{11 \times 145}{12} = 133$ lb/S.F.

live load 250

beams and Girder and other 17

400 lb/sf. 250 K/sf After reduction



Col. {JF - 8a, HD - 8a} = 16.67' × 53.5' × ~~400~~ 250 = 356.74 K = 222.96 K

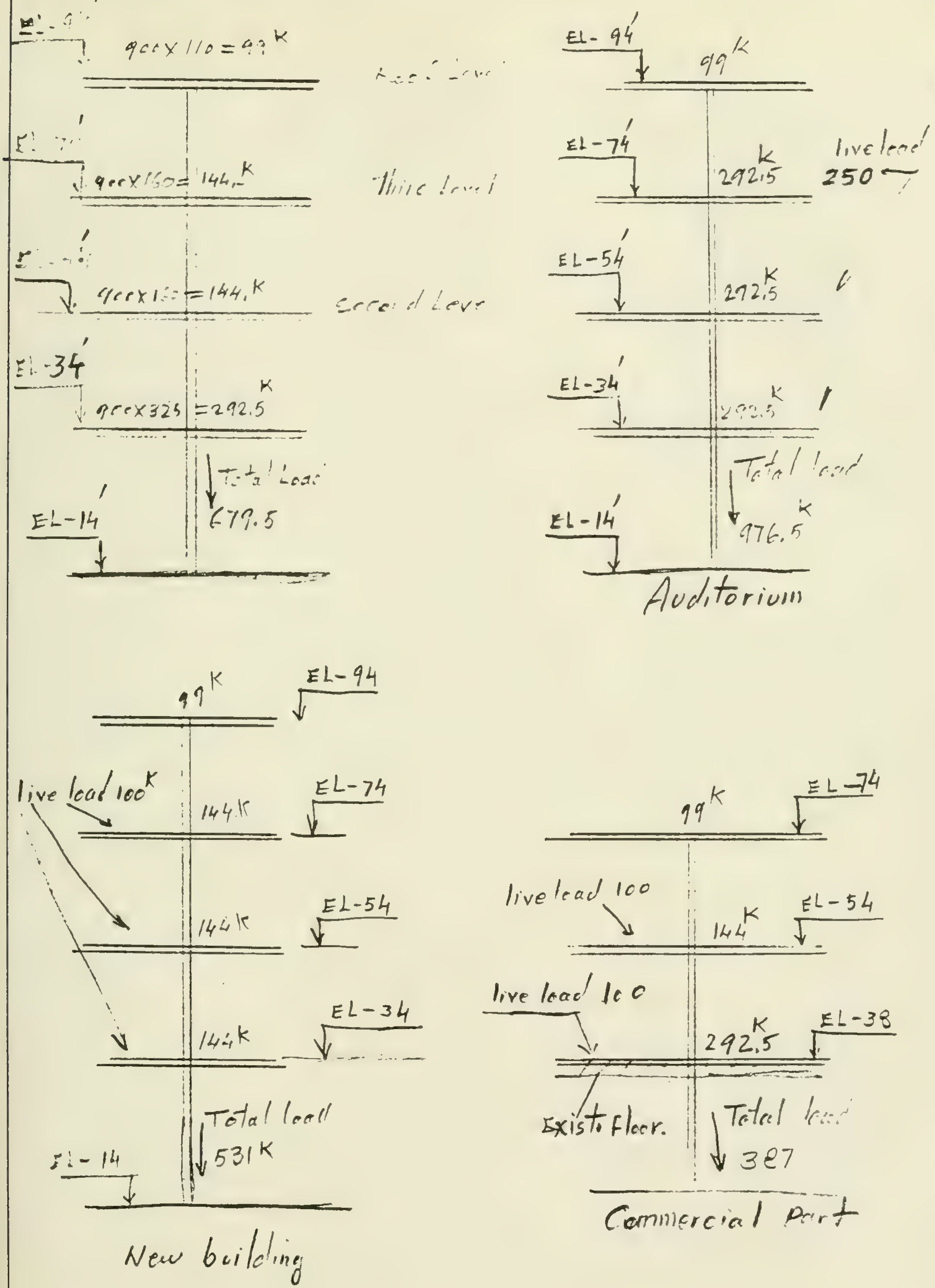
Col. {HD - 10, JF - 10} = (16.67'+15') × 53.5 × ~~400~~ 250 = 677.34 K = 423.59

Col. {HD - 11, JF - 11} = 30 × 53.5 × ~~400~~ 250 = 642. K = 401.25

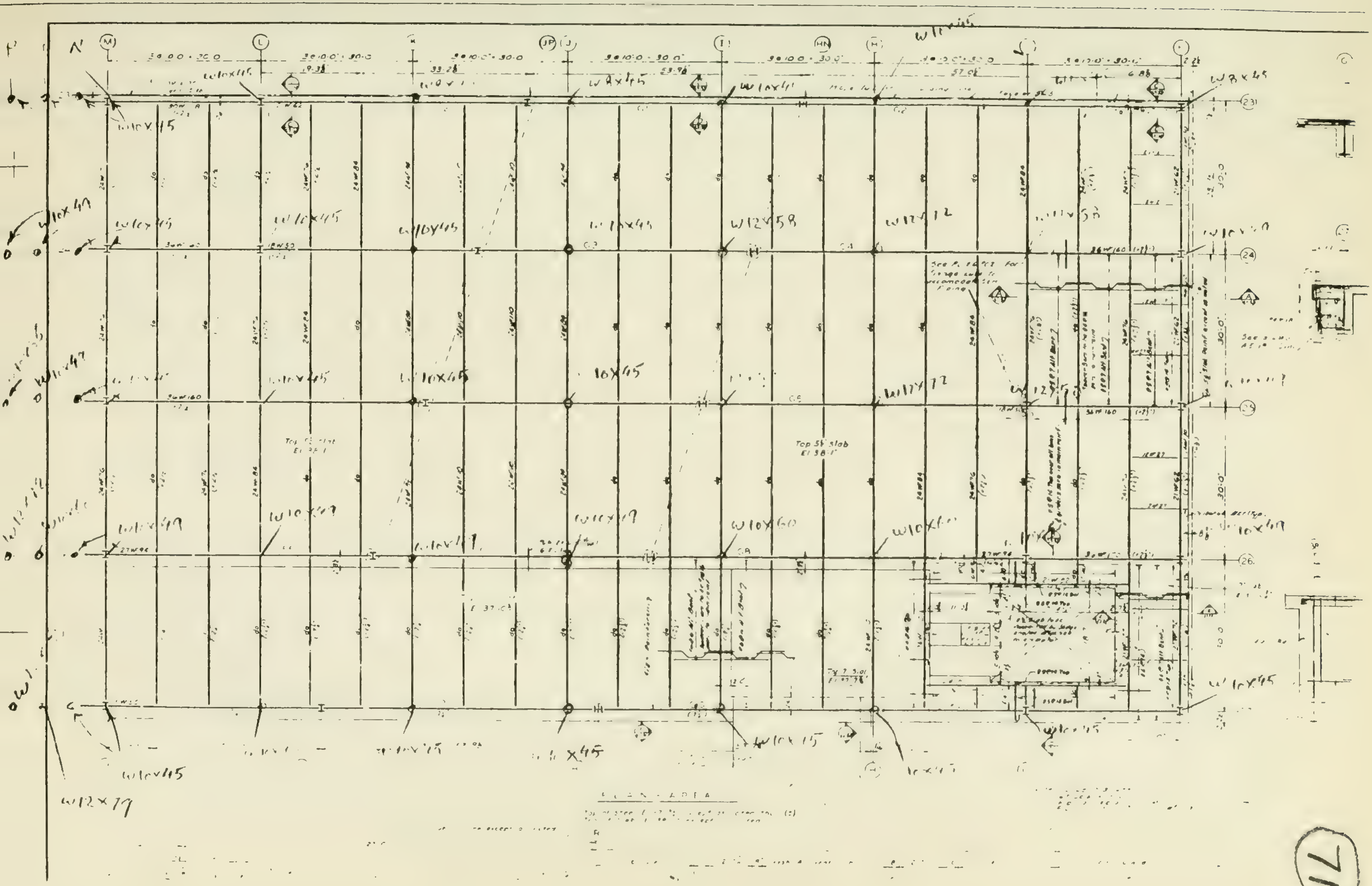
Col. HD - 12 = 2×15 × 53.5 × ~~400~~ 250 = 642 K. (Existing Roof of Turnpike) ≈ 401.25

Plaza Sub. Commercial

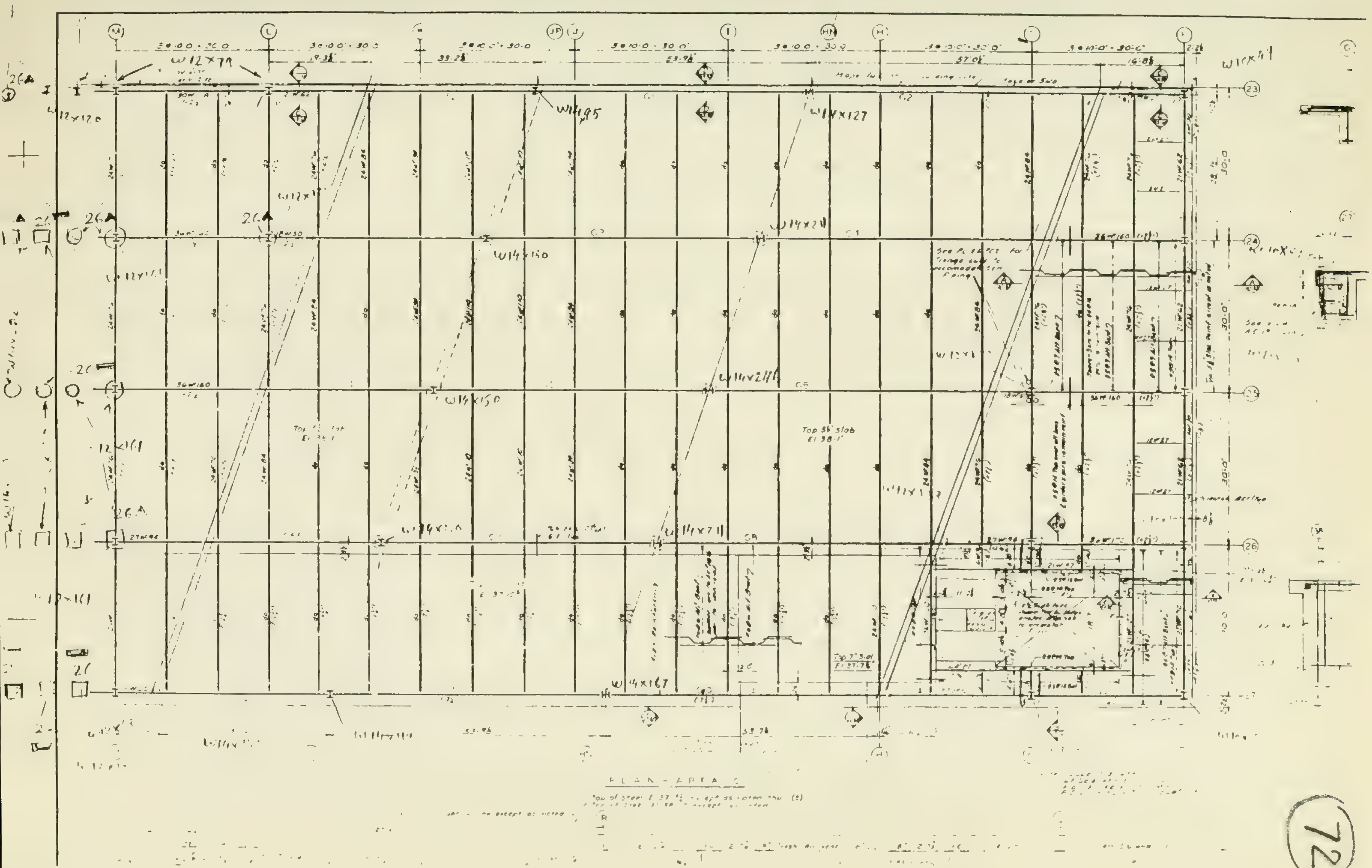
70



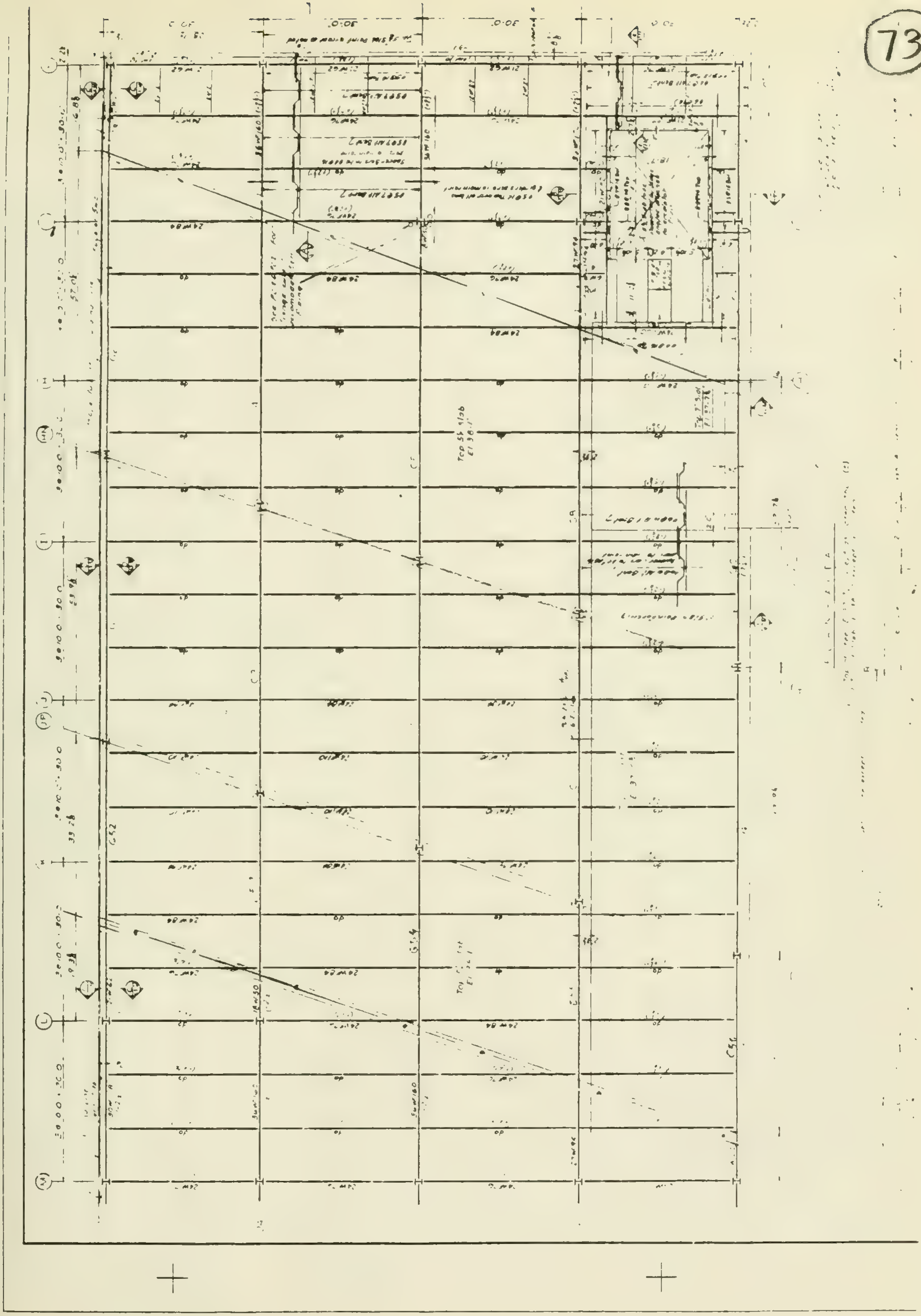
Commercial Center's Col.

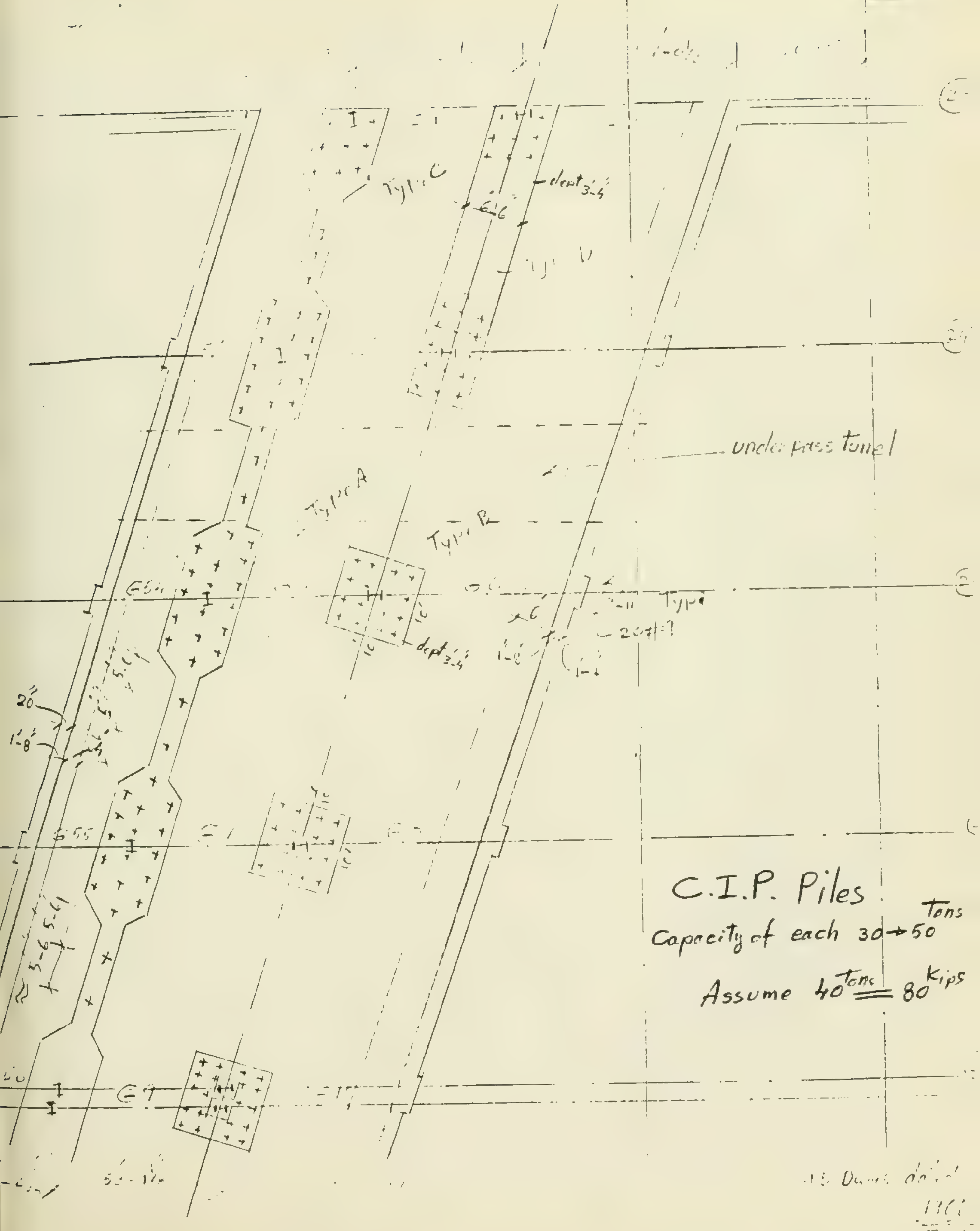


steel cal. 55.9



72





C.I.P. Piles
Capacity of each 30→50 ^{Tons}
Assume $40^{tons} = 80^{Kips}$

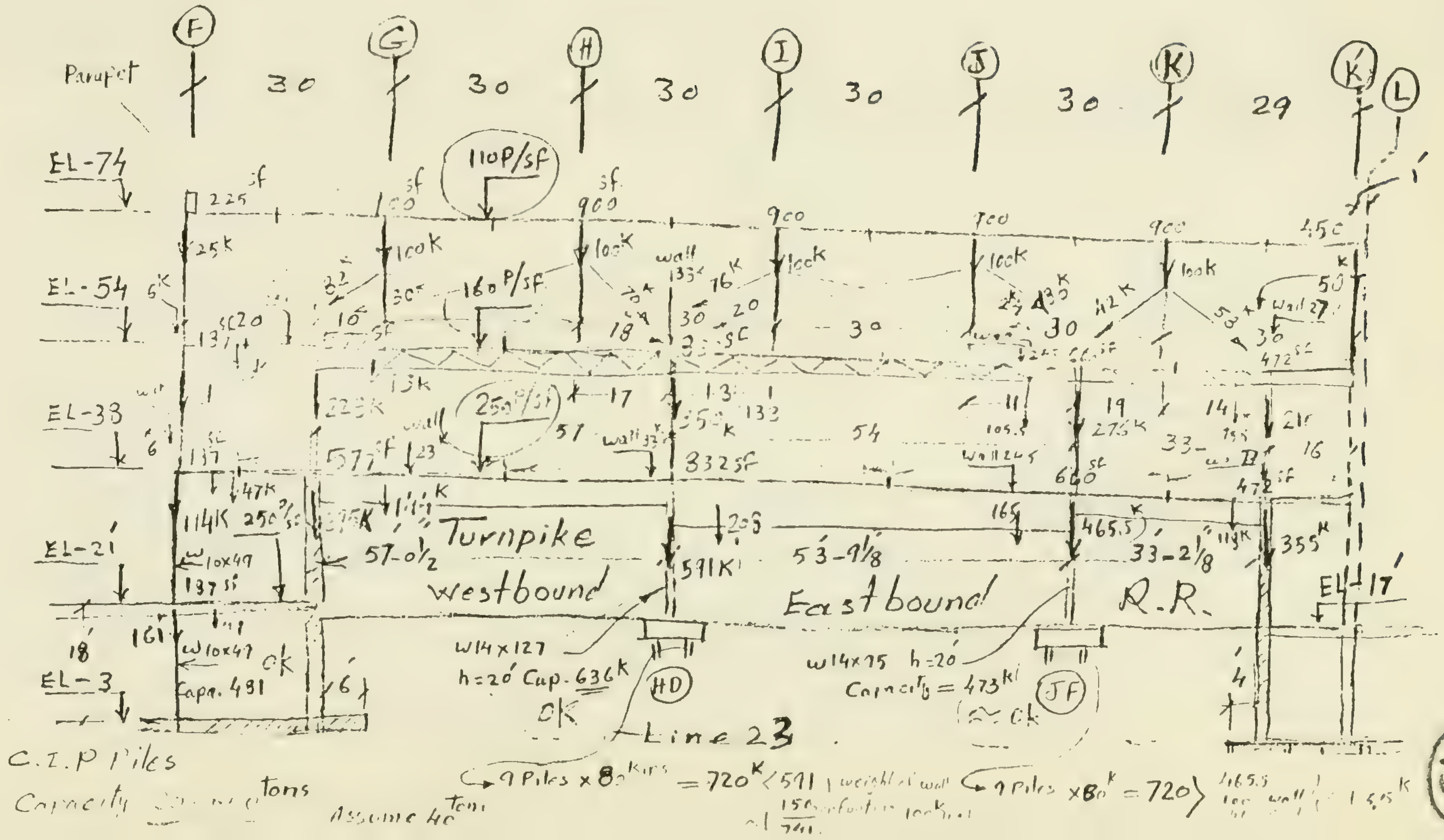
S-2.9 , S-2.8 , S-2-1

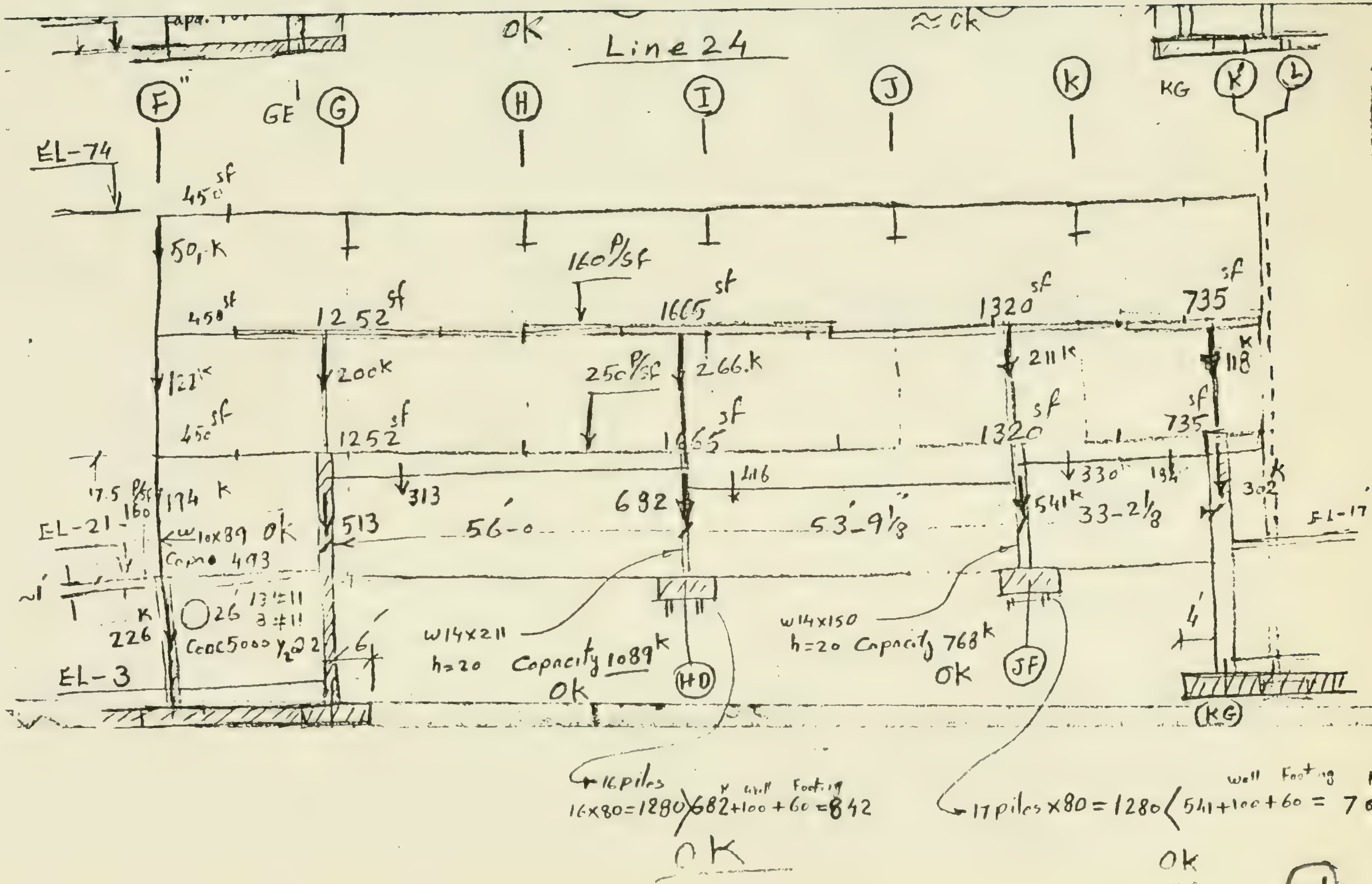
Commercial part above the turnpike

Total L. { Roof $110\ ^{P}/_{sf}$
Floor $160\ ^{P}/_{sf}$ at EL-54' and $250\ ^{P}/_{sf}$ for EL-38'

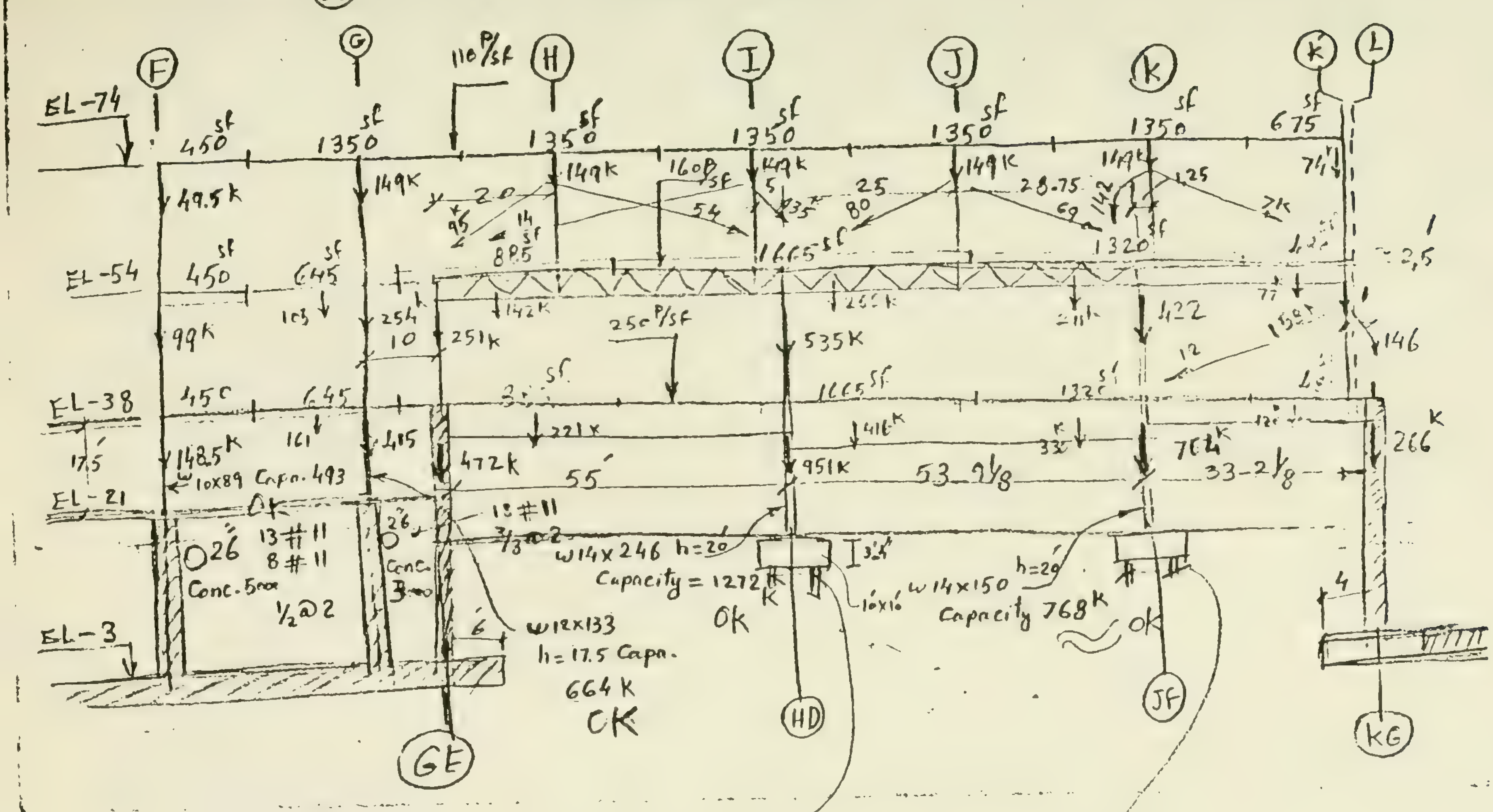
Line 23

sidewall = $600\ ^{P}/_{F}$ for 20' hight
Parapet = $20\ ^{P}/_{F}$



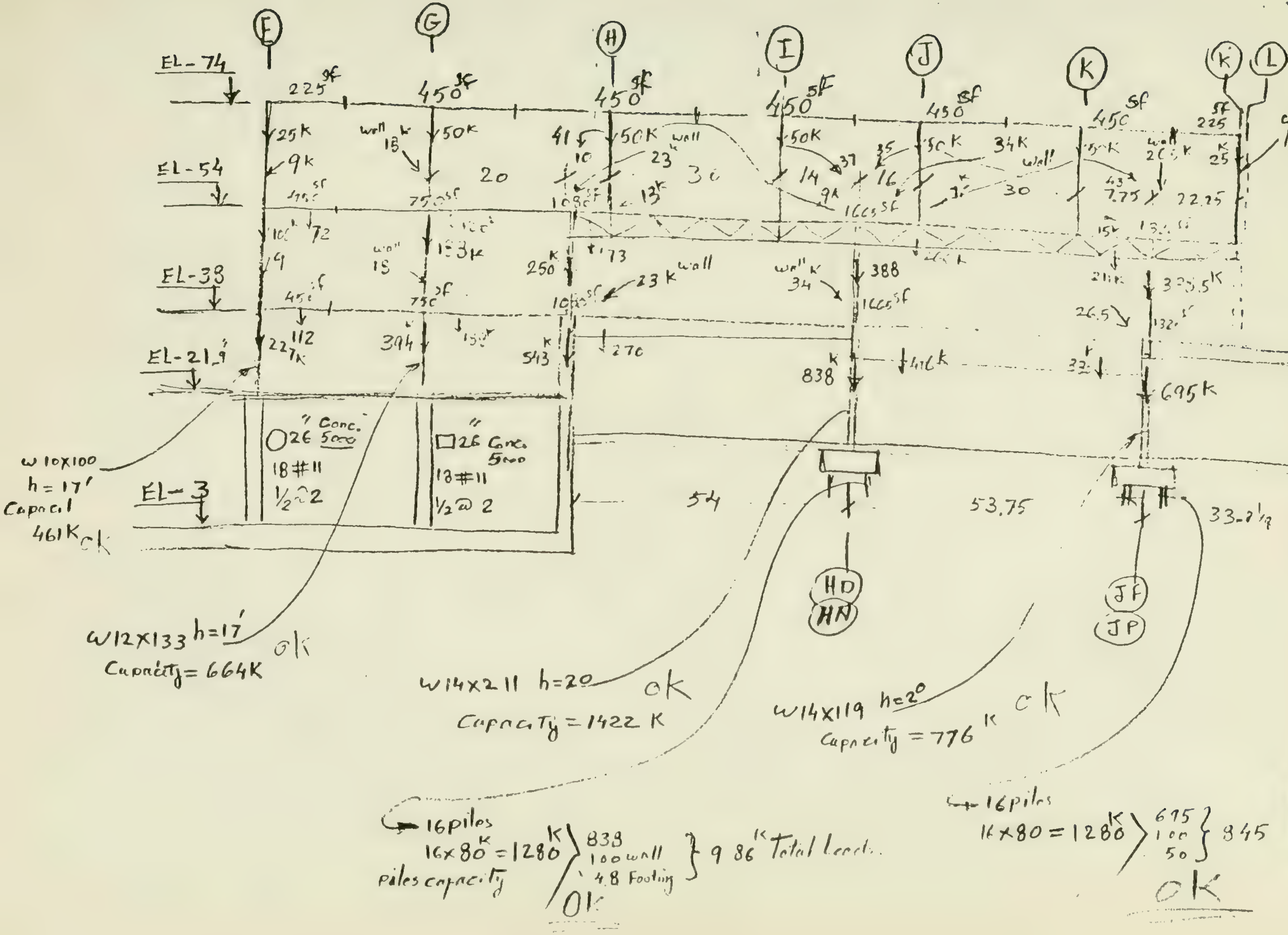

Line 24
OK
≈ OK
F
GE
G
H
I
J
K
KG
K'
L
EL-74
450 sf
50.1 K
160 p/sf
450 sf
1252 sf
1665 sf
1320 sf
735 sf
121 K
200 K
266 K
211 K
118 K
250 p/sf
450 sf
1252 sf
1665 sf
1320 sf
735 sf
17.5 p/sf
160
174 K
313
513
692
416
541 K
330
134
302 K
56'-0
53'-9 1/8
33-2 1/8
EL-21
W10x89 OK
Comp 493
EL-17
226 K
26
13 #11
3 #11
Conc 5000 1/2 @ 22
6'
4'
EL-3
W14x211
h=20
Capacity 1089 K
OK
HD
W14x150
h=20 Capacity 768 K
OK
JF
KG
16 piles
16x80=1280
K wall footing
682+100+60=842
OK
17 piles x 80 = 1280
wall footing
541+100+60 = 7
OK

Line 25



16 piles x 80 K = 1280 K > {951, 100, 48} 1100 K
Piles capacity
OK

16x80 = 1280 K > {764, 100, 48} 912 K
Piles capacity
OK

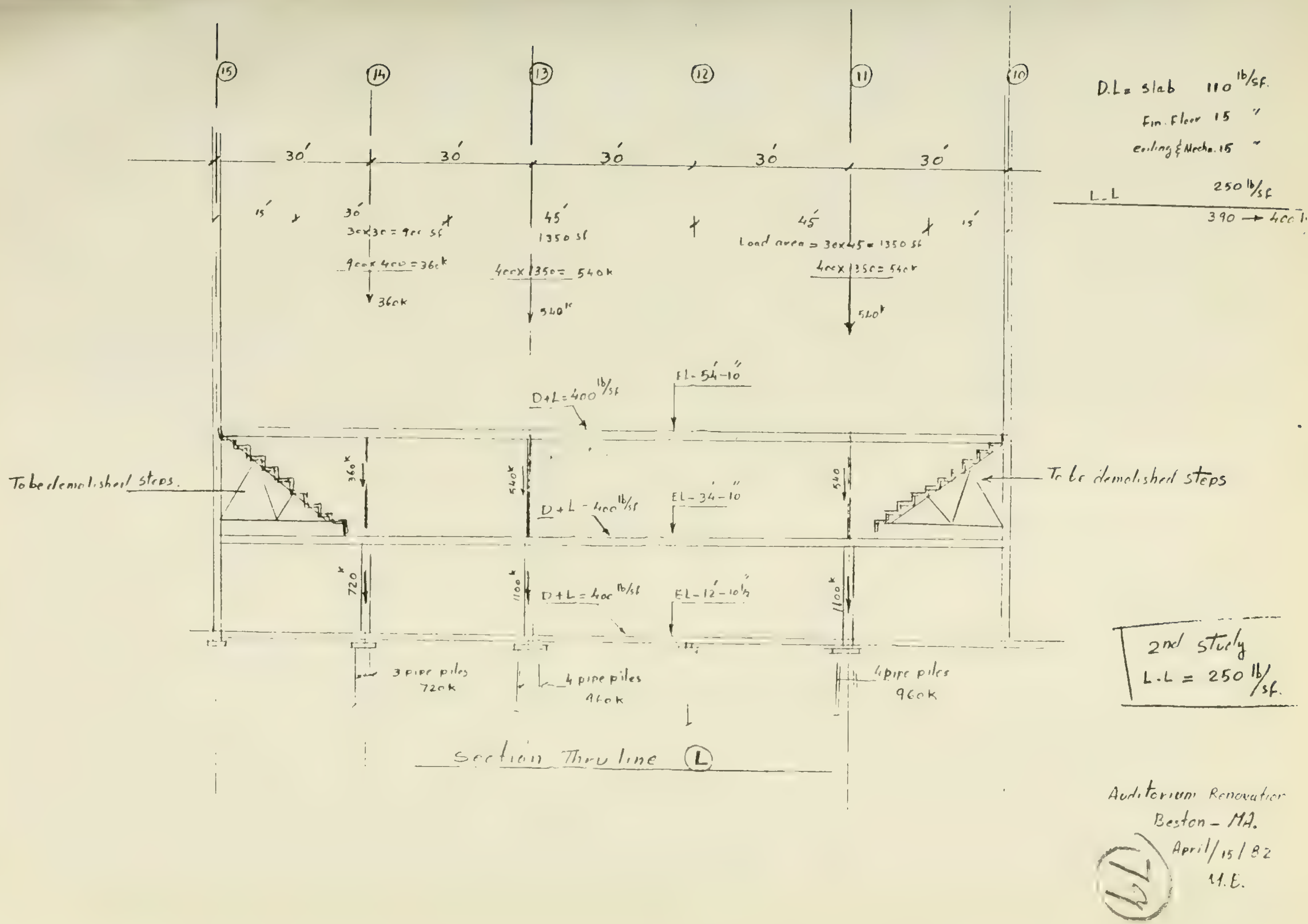


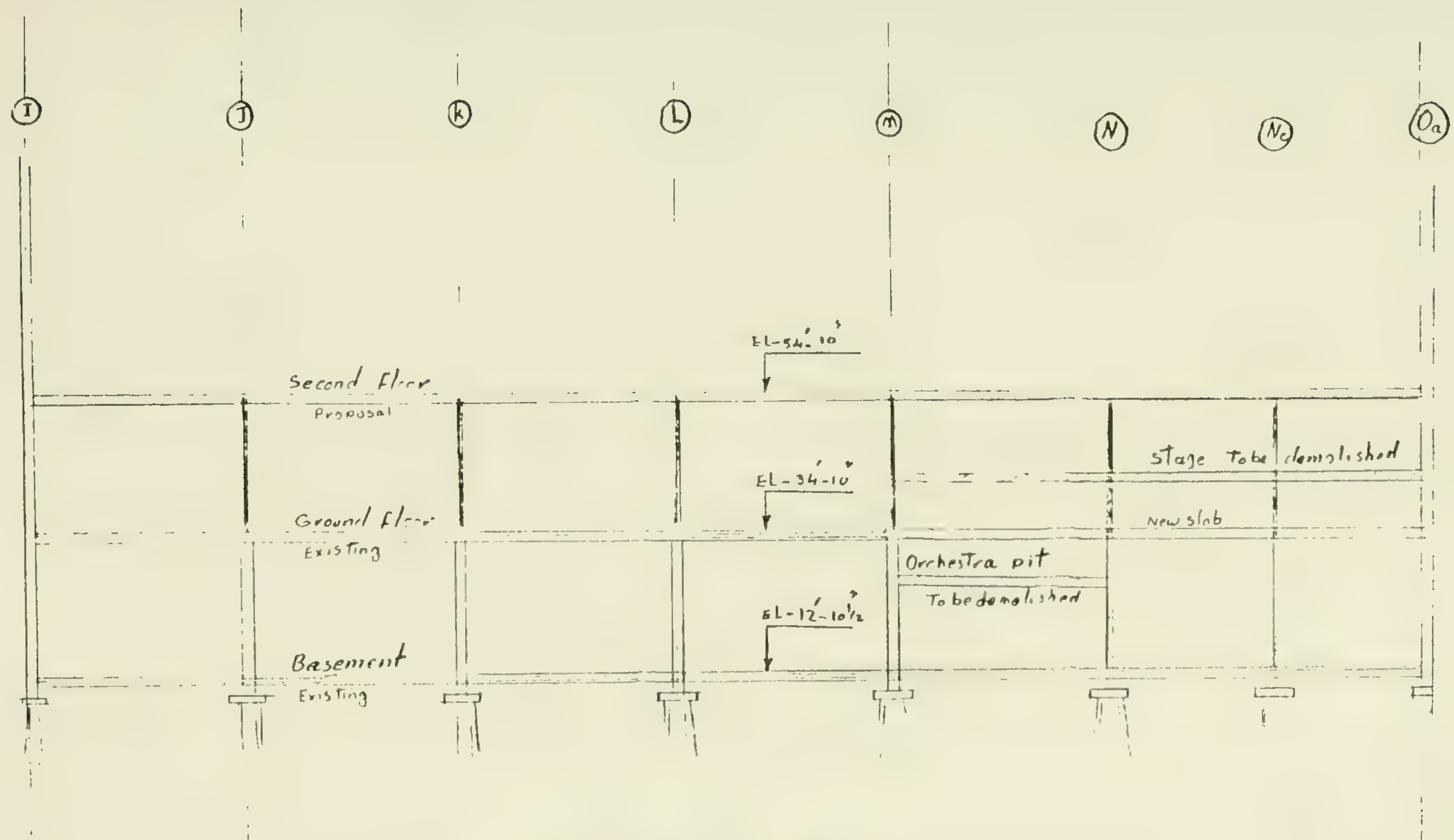
F
G
H
I
J
K
L
EL-74
EL-54
EL-38
EL-21.9
EL-3
225 sf
450 sf
25K
50K
wall 18K
O26 Conc. 5000
18#11
1/2@2
□26 Conc. 5000
18#11
1/2@2
838
695K
54
53.75
HD
HN
JF
JP
W10X100
h = 17'
Capacit
461K ok
W12X133 h=17'
Capacity = 664K ok
W14X211 h=20
Capacity = 1422 K ok
W14X119 h=20
Capacity = 776K ok
16piles
16x80K = 1280K
piles capacity
838
100 wall
48 Footing
986K Total Loads
OK
16piles
16x80 = 1280K
695
100
50
845
ok

Structural Calculations

Auditorium Infill

Weidlinger Associates


15
14
13
12
11
10
30'
30'
30'
30'
30'
15'
30'
45'
45'
15'
3ox30 = 900 sf
900x400 = 360k
360k
1350 sf
400x1350 = 540k
540k
Load area = 30x45 = 1350 sf
400x1350 = 540k
540k
D.L = slab 110 lb/sf.
Fin. Floor 15 "
ceiling & Mecha. 15 "
L.L 250 lb/sf
390 → 400 l.
EL-54'-10"
D+L=400 lb/sf
To be demolished steps.
To be demolished steps
360k
540k
540
D+L - 400 lb/sf
EL-34-10
720k
1100k
1100k
D+L = 400 lb/sf
EL-12'-10½"
3 pipe piles
720k
4 pipe piles
960k
4 pipe piles
960k
section Thru line L
2nd Study
L.L = 250 lb/sf.
Auditorium Renovation
Boston - MA.
April/15/82
M.E.
77



Longitudinal section Thru Line (11)

2nd study
L.L. = 250 lb/SF

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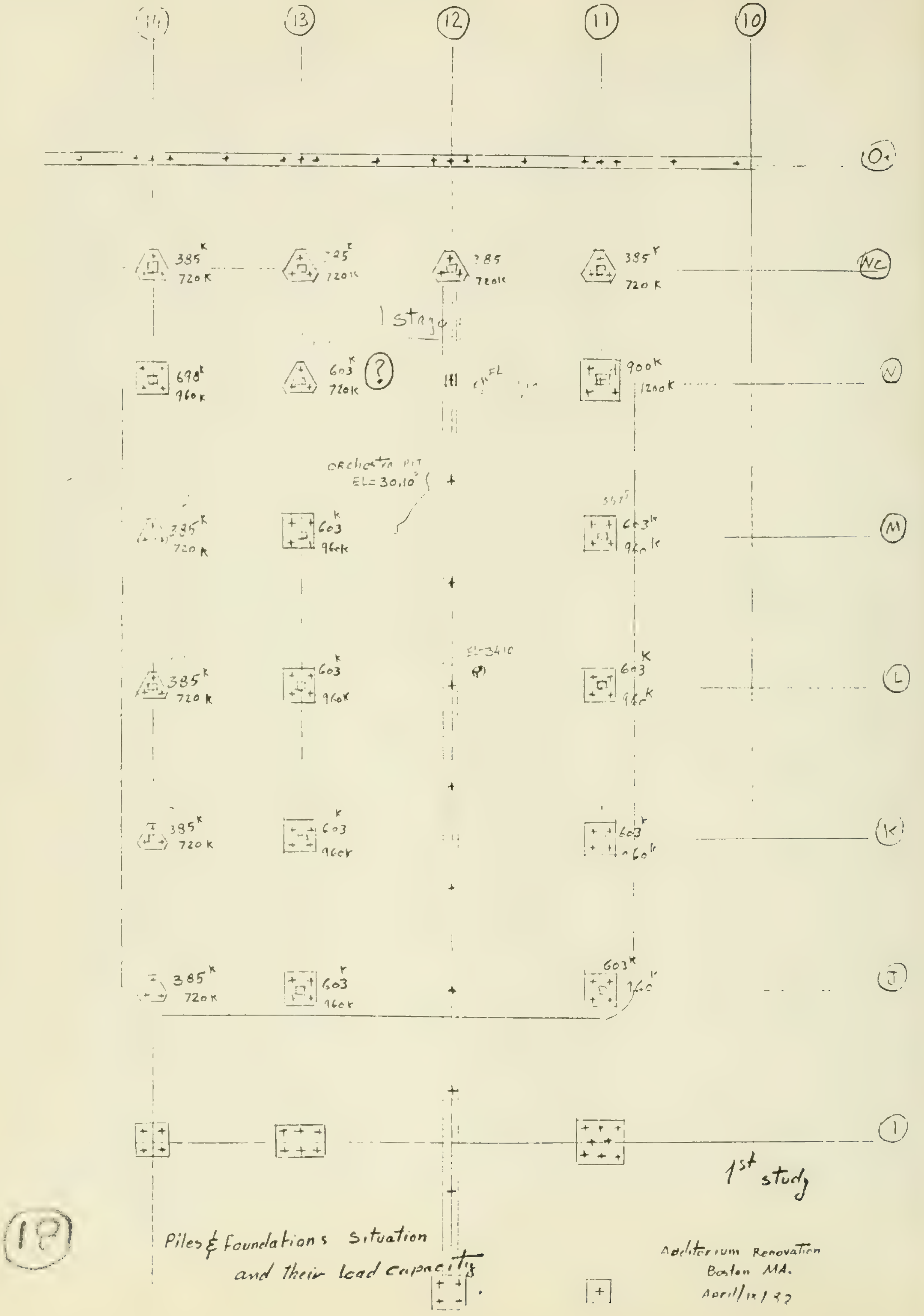

14
13
12
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O
NC
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M
L
K
J
I
385K 720K
25K 720K
385 720K
385K 720K
stage
698K 960K
603K 720K
?
900K 1200K
orchestra pit
EL=30,10'
385K 720K
603K 960K
603K 960K
EL-34.10
385K 720K
603K 960K
603K 960K
385K 720K
603K 960K
603K 960K
385K 720K
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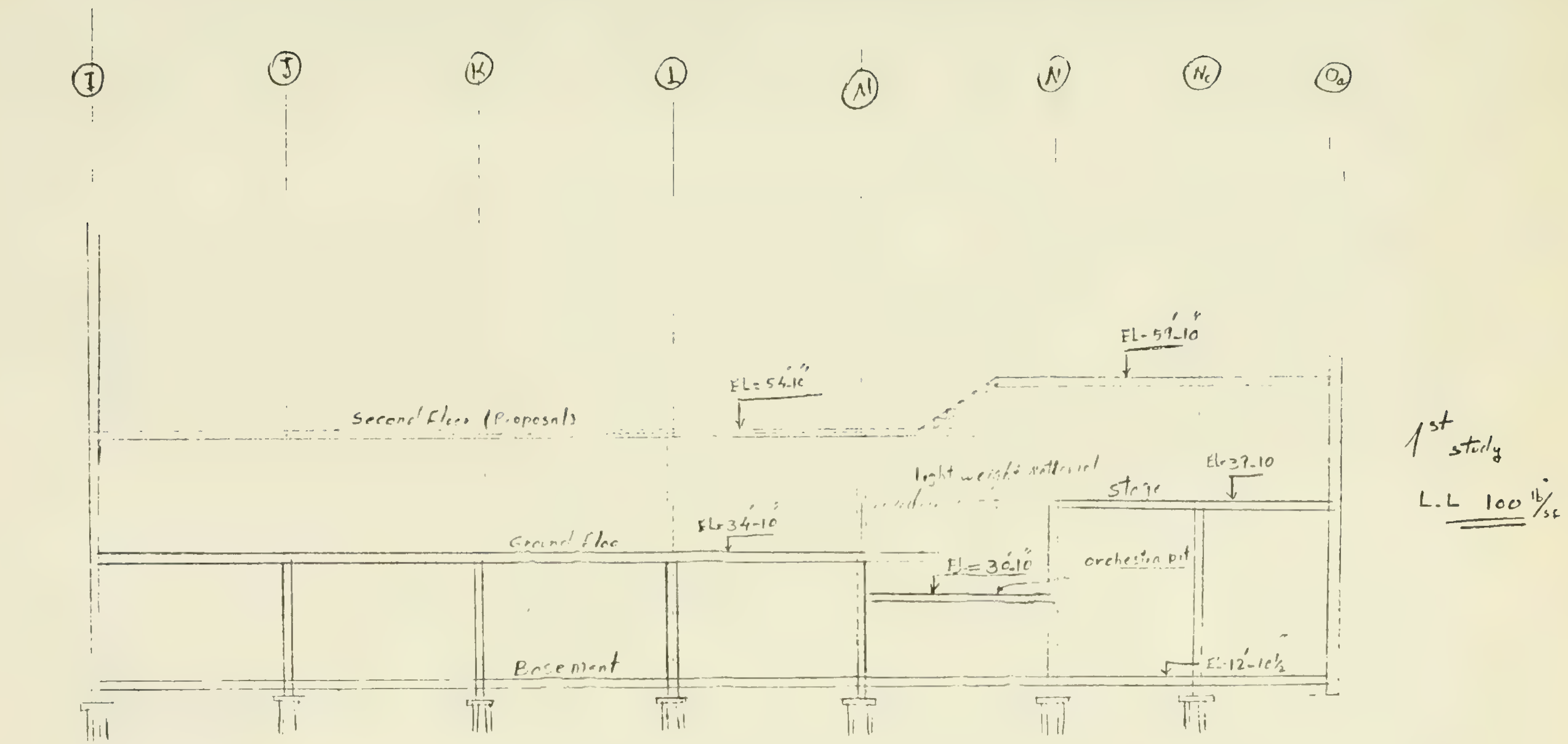
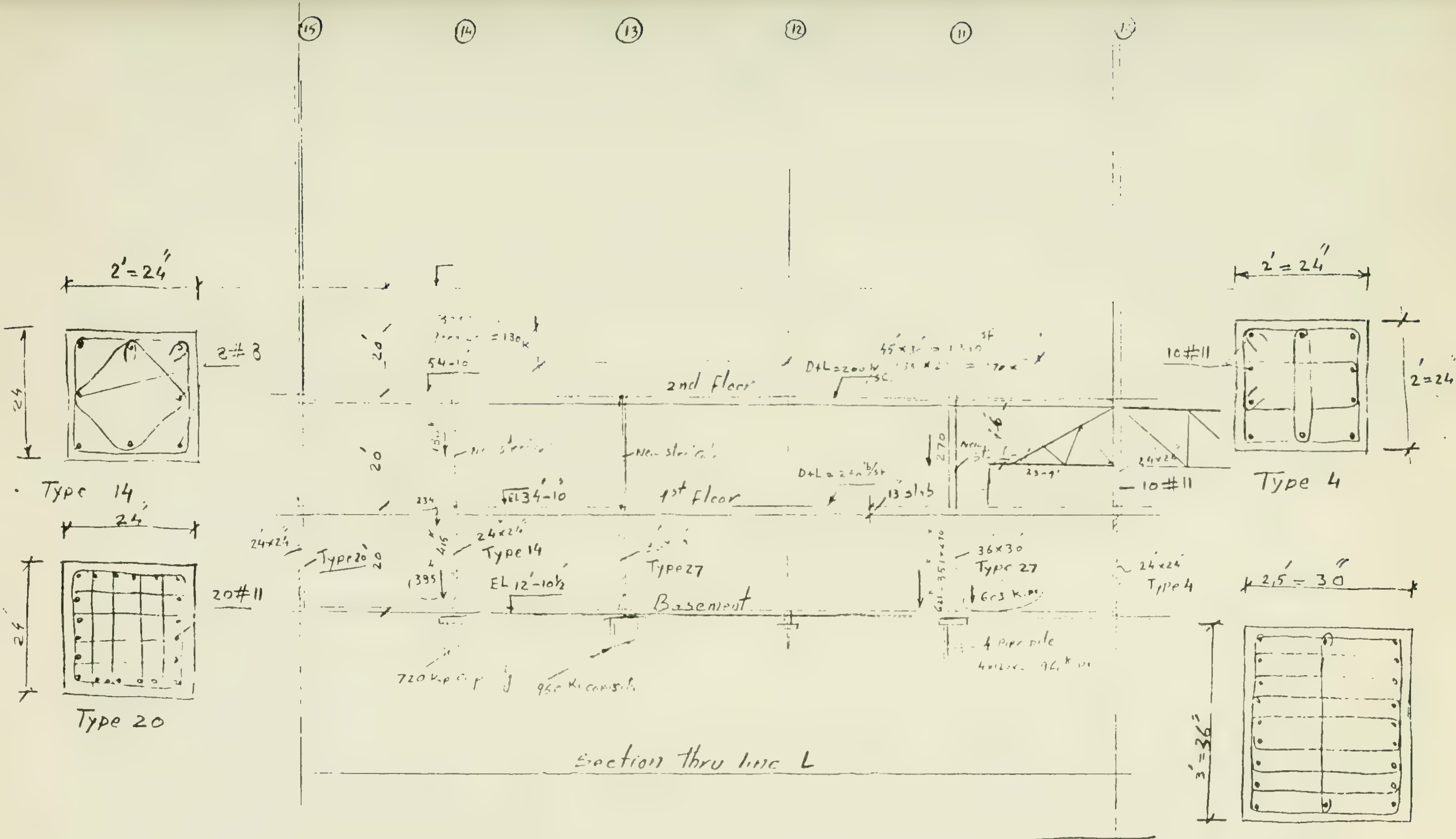
603K 960K
1st study
18
Piles & Foundations situation
and their load capacity
Auditorium Renovation
Boston MA.
April/1x/82



Auditorium Extension
Boston. MA
April/13/82
M.B

82



1st study
L-L = 100 lb/sf

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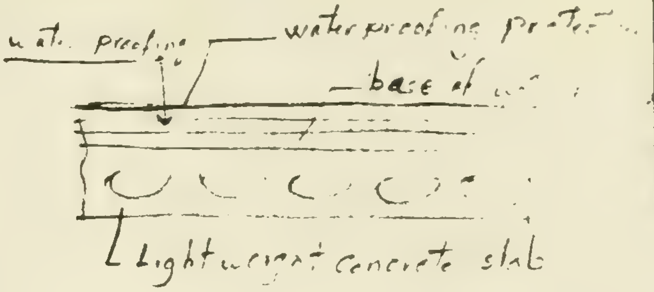
Boston, MA.

83

Structural Calculations

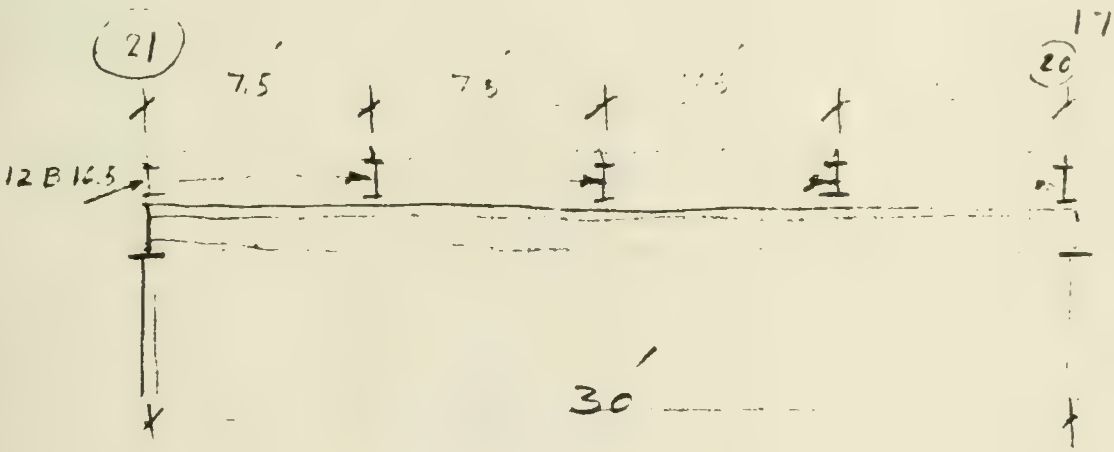
Roof Over Existing Hynes Auditorium

Weidlinger Associates



Load

Light weight concrete slab (plank) 3" Aprox.	25	P/sf
Topping (base of water proofing)	10	"
water proofing	3	"
protection of water proofing	15	"
Beams	5	
	58	P/sf
if L.L.	100	
Mechanical and ceiling	12	"
	170	P/sf



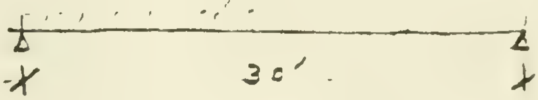
Typical section

Auditorium checking of roof between lines 19 to ...

(85)

Beams:

$w = 170 \times 7.5 = 1275$



$$M = \frac{wl^2}{8} = \frac{1275 \times 30 \times 30}{8} = 143.44 \text{ k/ft.}$$

$$\sigma = 22 \text{ k/in}^2$$

$$\text{Requ. } S = \frac{143.44 \times 12}{22} = 78.2 \text{ in}^3$$

take W 18×46 $\quad S_x = 78.8 \text{ in}^3$

Auditorium checking of roof between lines [illegible]

(86)

w 12 x 16 $\quad S_x = 17.1 \text{ in}^3$

$M = \frac{17.1 \times 22}{12} = 31.35$ K.F

$w = \frac{M \times 8}{l^2} = \frac{31.35 \times 8 \times 1000}{400} = 627.$ #/'

Total Load (D.L + L.L) $= \frac{627}{7.5} = 83.6$ #/sf

$M = \frac{wl^2}{10} = 43$ K.F

$w = \frac{43 \times 10}{900} = 0.477$ Kips

$\text{Total L.}/sf = \frac{477}{7.5} = \underline{63}$ P

D.L	Take slab	20 P
	Gravel	10
	water pr.	3
	Mecha. & Ceiling	—
		33

L-L $\quad 63 - 33 = \underline{\underline{30}}$ P/sf

SNOW	30	
SLAB	20	
GRAVEL	10	
W. P.	~~5~~ 3	
Ceiling	~~5~~ 0	63
	65 – 70	

(63)

Beams with span 30'

$\frac{30}{4} = 7.5'$

7.5

$w = 7.5 \times 75 = 562.5$ lb/f lin

$M = \frac{562.5 \times 30^2}{8} = 63$ k'

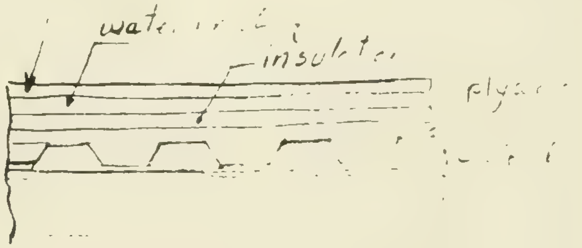
$S_x = \frac{63.28 \times 12}{22} = 34.51$ in^3

take W 14 x 26 S = 35.3 in^3

26 ÷ 7.5 = weight 3.47 lb/sf

water proof protection

Roof detail.



Roof deck Type "BA"

(Gage 18)

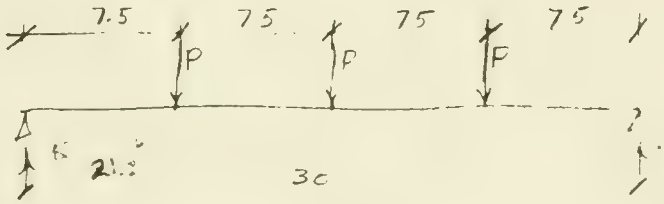
Two spans deck

All materials of roofing	25	P/sf
Beams	5	
Mechanical & ceiling etc.	15	
Snow	30	
	75	

30
6
6

(87)

Girders with Span 30'



$P = .75 \times 30 \times 75 = 16.88$ kips

$R = \frac{3P}{2} = \frac{3 \times 16.88}{2} = 25.32^{K}$

$M_{Max} = 25.32 \times 15 - 16.88 \times 7.5 = 253.20$ K F

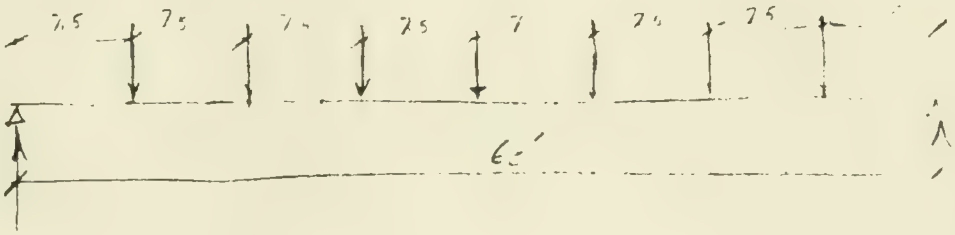
Required $S_x = \frac{253.20}{\ldots} = 138.11 \text{ in}^3$

take W 21×68 $S = 140.\text{in}^3$

steel weight per s.f. $\frac{68}{30} = 2.26$ #/sf

90

Trusses with span 60'



$$w = \text{uniform load} = \frac{7 \times 7.5 \times 75 \times 30}{60} = 1.969 \text{ K/lin. ft}$$

$$M_{Max} = \frac{1.969 \times 60^2}{8} = 886 \text{ K.ft.}$$

$$M = 886 \times 12 = 10\,632 \text{ K.in}$$

Force in top and bottom of truss $\frac{10\,632}{42} = 253.2$

Section area of steel $\frac{253.2}{22} = 11.50 \text{ in}^2$

Take WT 10.5 × 41.5 area $= 12.2 \text{ in}^2$

W. of Truss: $2 \times 41.5 \times 1.30 = 107.9$ P/F linear

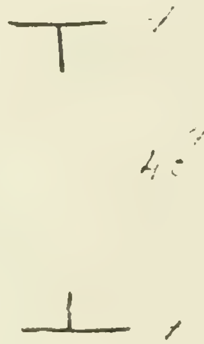
weight per s.f. $\frac{107.9}{30} = \boxed{3.59}$ P/sf.

(91)

Trusses with span 90'

w = uniform load = $\frac{11 \times 7.5 \times 7.5 \times}{90}$ = 2.06 k/ft

$M_{max} = \frac{2.06 \times 90^2}{8} = 2086$ K.F.



Required section area $\frac{2086 \times 12}{48 \times 22} = 23.70\ in^2$

take WT 18 × 85 $A = 25''$

W.o. Truss 2 × 85 × 1.30 = 221 P/F linear

weight per s.f. $\frac{221}{30}$ = 7.36 P/sf

(92)

Load on Col.

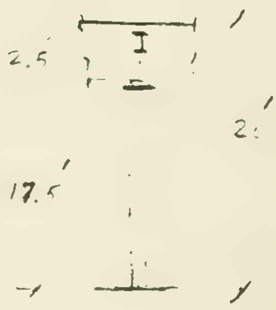
$30 \times 30 \times 75^{lb} = 67.5$ K take W8X28

$30 \times 45 \times 75 = 10[illegible]$ K " W8X31

$30 \times 60 \times 75^{lb} = 135$ " " W8X40

$\frac{90}{2} + \frac{60}{2} = 75$

$30 \times 75 \times 75^{lb} = 168.75$ " " W8X48



weight of col per s.f. $= \frac{28}{900} = 0.03$ P/sf

" " " " $\frac{40}{180} = 0.0[illegible]$

" " " " $\frac{48}{225} = 0.02$

Total weight of steel for roof

Per S.F.

Span	Beam	Girder Truss	col	Total weight
30x30	3.47	2.26	0.5	5.76
60x30	3.47	3.69	0.4	7.08
90x30	3.47	7.36	0.4	10.85

average of using steel

$\frac{8.5 \times 5.76 + 7.08 \times 2}{10.5} = 6.01$ P/sf

$\frac{7.5 \times 5.76 + 10.85 \times 3}{10.5} = 7.21$ "

Midal 6.61 P/sf.

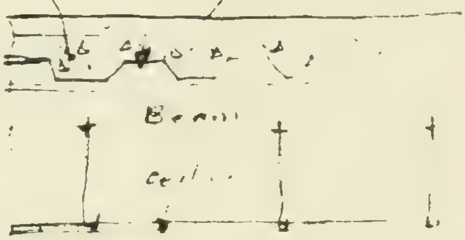
93

Third floor (New)

1½ B-lok Deck
Gage 22

If unshored is required
Take gage 18 or 19

$t = 4$ [illegible]



Loads:

Dead [illegible] slab 4 concrete + deck   50
Beam   5
[illegible] & other Ceiling   10

[illegible]5

[illegible] load   100

Total load   165

Effective [illegible]

$b = \frac{1}{4}$ [illegible]

$b =$ [illegible]

$b = 16 \times t + [illegible] = 16 \times 4 + 6 = 72'$

$b = 72''$ Gov[illegible]

$n = 9$

$b/n = 8$

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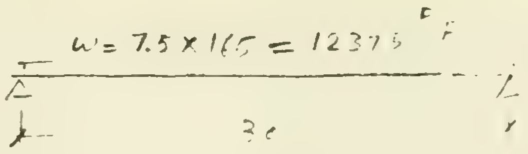
Auditorium Easton, Md. Third Floor

(94)

Third Floor (New)

$F'_c = 4000\ \#/in^2$

$F_y = 36000\ \#/in^2$



$M = \frac{1237.5 \times 900}{8} = 139.22$ K.F.

Required section modulus $\frac{139.22 \times 12}{22} = 75.93\ in^3$

See page 60 (steel design file – Bethlehem)

take W 16 × 40 with $S_x = 65\ in^3$ $\frac{40}{72} =$ [illegible]

See page 75 composite of W16x40 ...

$A = 11.77\ in^2$ $S_b = 92.7\ in^3$ $S_t = 2050\ in^3$ $S_c = 302\ in^3$ $I = 1418\ in^4$ [illegible]

Stresses $\begin{cases} \text{bottom fibers} = \frac{M}{S} = \frac{139.22 \times 12}{92.7} = 18.0 < 22 \\ \text{Top fibers} = \frac{M}{S \times n} = \frac{139.22 \times 12}{302 \times 9} = 0.61 < 0.45 f'_c \end{cases}$

$\Delta = \frac{ML^2}{160 I} = \frac{139.22 \times 30^2}{160 \times 1418} = 0.55 < \frac{L}{300} = \frac{360}{300} = 1.2''$ ok

Horizontal shear

$\begin{cases} \text{Concrete shear} = \frac{0.85 f'_c \times A_c}{2} = C \\ \text{steel shear} = \frac{A_s \times F_y}{2} = S \end{cases}$

$C = \frac{0.85 \times 4 \times 72 \times 4}{2} = 244.8$ $S = 211.86$

See page 5-39 AISC

use $\frac{1}{2}\ \phi\ 3''$ stud $q = 13.3$ $N = \frac{211.86}{13.3} = 16$ connectors

Girders:

(95)

$P = 7.5 \times [illegible]5 \times [illegible] = 37.12$ k

$[illegible] = \frac{3 \times [illegible]}{[illegible]} = [illegible]$

$R = \frac{3P}{2} = \frac{3 \times [illegible]}{2} = 55.69$ k

$M_{max} = 55.6[illegible] \times [illegible] - 37.[illegible] \times [illegible] = [illegible].95$ K. F

Required Modulus section $= \frac{[illegible]}{[illegible]} = [illegible]$

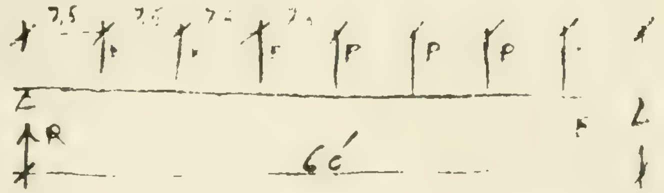
Take [illegible] $S_x = 329$ [illegible]

Girder weight per s.f = [illegible] [illegible] #/s.f

Beam weight per s.f. $= \frac{[illegible]}{7.5} = 5.33$

For area 30×30 weight of steel = 9.20 #/s.f

Estimation Truss with 60' span.



$P = 75 \times 165 \times 30 = 37.12$ Kips

$R = \frac{7 \times 37.12}{2} = 129.92$

w = uniform load $= \frac{129.92}{30} = 4.33$ Kip/F.

$M = \frac{4.33 \times 3600}{8} = 1948.5$ K.F.

$M = d \times A_s \times \sigma_s = 1948.5$ $\sigma_s = 22$ K/in²

$d = 42$

$A_s = \frac{1948.5 \times 12}{22 \times 42} = 25.30 \text{ in}^2$

take WT 15 × 86.5 $A_s = 25.4 \text{ in}^2$

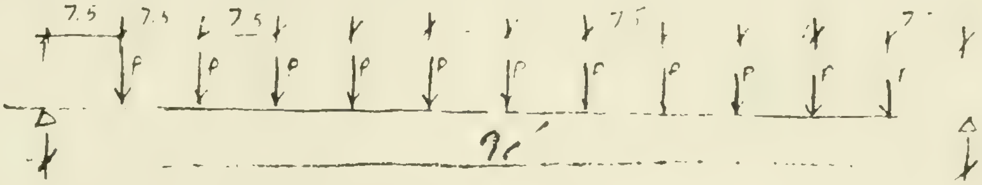
Truss weight/F $2 \times 86.5 \times 1.30 = 224.9$

unit weight $\frac{224.9}{30} = 7.49$ #/S.F.
Beam " See page 7   5.33 "   } 12.82

$\boxed{\text{It isn't Economic}}$

also Deflection is too much

Estimation Truss with 90' span



(97)

$P = 7.5 \times 165 \times 30 = 37.12$ Kips

$w = \frac{11 \times 37.12}{90} = 4.53$ K/F linear uniform load

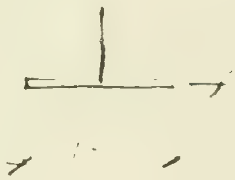
$M_{max} = \frac{4.53 \times 90^2}{8} = 4586.62$ K.F

$\sigma_v = 22$ K/

$A_s = \frac{M}{d \times \sigma_v} = \frac{4586.62 \times 12}{48 \times 22} = 52.12$ in²

$d = 4'$

[illegible] $3 \times 15 =$ [illegible]



[illegible] $\frac{4 \ldots \times 3 \times \ldots}{1\ldots} = 133$ P/[illegible]

weight Joist [illegible] = 477.75

unit weight $\frac{477.75}{30} = 15.92$
Beam " [illegible] I 5.33 } 21.25 P/s

It is not economic

Also Deflection is Too much

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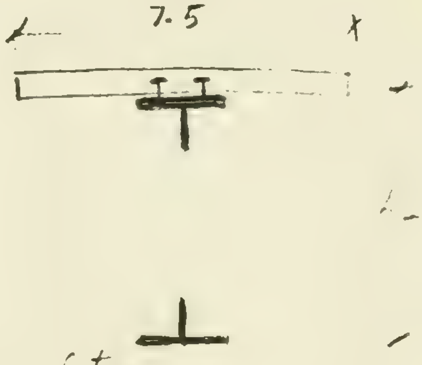
Auditorium Boston, MA. "Addition" Thir...

Composite Truss with 60' span as a Joist

Deck 1½ B-lok gage 22



$F'_c = 4$ ksi

$F_y = 36$ k/in²

$w = 7.5 \times 165 = 1237.5$ P/F linear of truss

$M = \dfrac{1237.5 \times 60^2}{8} = 556.88$ F.k.

required $S_x = \dfrac{556.88 \times 12}{22} = 303.75$

area $\dfrac{303.75}{42} = 6.33$ in²

WT 9 × 23 area 6.77

weight of Truss = 2 × 23 × 1.30 = 59.8 P/F

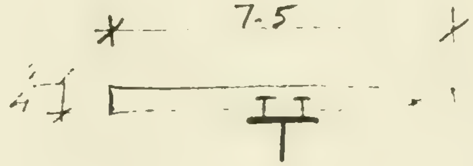
unit weight $\dfrac{59.8}{7.5} = 7.97$ P/F

For deflection see page 17

Composite Truss with 90' span as a Joist

Deck 1½ B-lok gage 22

(77)



$f'_c = 4$ K/in²

$F_y = 36$ K/in²

w = 7.5 × 1.65 = 1237.5 #/ft linear of truss

$M = \frac{1237.5 \times 90^2}{8} = 1252$ 96 K.F.

required area of steel = $\frac{1252 \times 12}{[illegible]}$ = [illegible] in²

Take WT 13.5 × [illegible] $A_s = 15$ in²

weight of truss 2 × [illegible] × 1.3[illegible] = 13[illegible]6 [illegible]

unit weight [illegible] = 17.68 #/sf

See page ~~11~~ 13 deflection is too much

so take WT 15 × 58

go page 14

Auditorium Boston, Addition

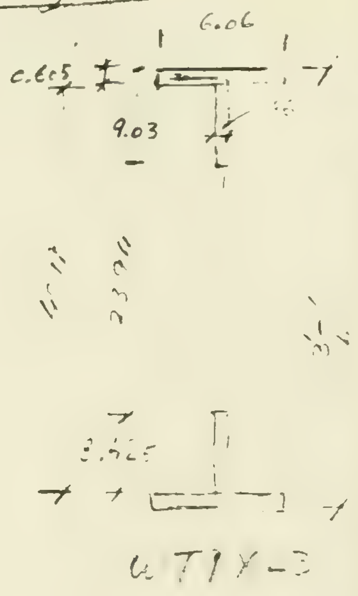
checking of Deflection Joist by 60' span

$I = I_f + I_w$

$I_f = \frac{6.06}{12}\left(42^3 - 40.79^3\right) = 3141.41 \; in^4$

$I_w = \frac{0.36}{12}\left(40.79^3 - 23.??^3\right) = 1624.40$

$I = 4765.81 \; in^4$



(100)

See page 11 $M = 556.85$ F.k

$\Delta = \frac{ML^2}{160 I} = \frac{556.85 \times 60^2}{160 \times 4766} = 2.62 < \frac{60 \times 12}{250} = 2.88$

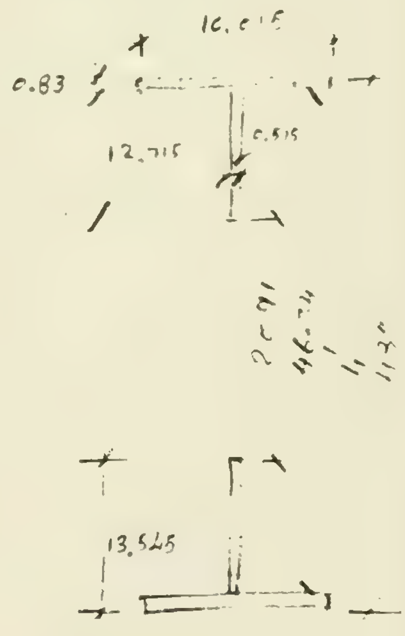
Checking of deflection Joist by 90' span

$I = I_f + I_w$

$I_f = \frac{16.015}{12}\left(48^3 - 46.34^3\right) = 9248.59$

$I_w = \frac{0.515}{12}\left(46.34^3 - 20.?^3\right) = 3878.29$

$I = 13126.88$



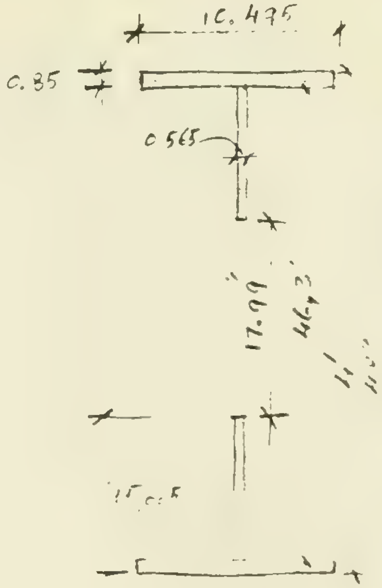
See page 12 $M = 1252.96$

$\Delta = \frac{1252.96 \times 90^2}{160 \times 13126.88} = 4.83 > \frac{90}{250} = 4.32$

WT 13.5 X 51

Must be stronger than this profile

check deflection Joist by 90' span



WT 15 x 58

$$I = I_f + I_w$$

$$I_f = \frac{10.495}{12}(48^3 - 46.3^3) = 9917.03$$

$$I_w = \frac{0.565}{12}(46.3^3 - 17.99^3) = 4399.02$$

$$I = 14316.05$$

$$\Delta = \frac{1252.96 \times 90^2}{160 \times 14316.05} = 4.43 \approx 4.32$$

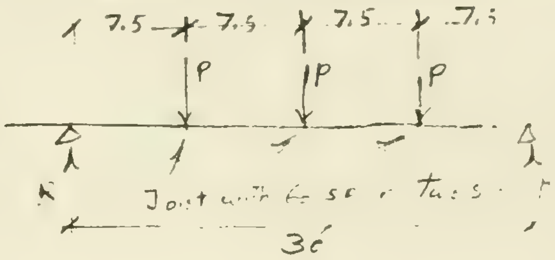
↪ for sol'n take WT 15 62

weight of Joist $= 2 \times 62 \times 1.30 = 161.2$ P/ft lin.

unit weight $\frac{161.2}{7.5} = 21.49$ P/sf

Girder for support of Joists with 60' span on two

102



$P = 7.5 \times 165 \times 30 \times 2 = 74.25$ Kips

$R = \frac{3P}{2} = \frac{3 \times 74.25}{2} = 111.375$ kips

$M = 111.375 \times 15 - 74.25 \times 7.5 = 1113.75$ F.k.

Required $S_x = \frac{1113.75 \times 12}{22} = 607.5\ in^3$

take W 36 X 182 $S_x = 623$

$\frac{182}{60} = 3.03$ #/sf.

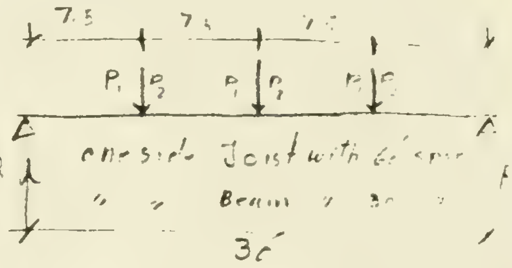
From page

weight of Joist 60' 7.97
" " Girder 3.03 } 11 #/sf

Auditorium Boston, MA. "Addition" Third Floor

Girder for support of Joist with 60 span one side

(103)



$P_1 = 7.5 \times 165 \times 15 = 18.562^{k}$

$P_2 = 7.5 \times 165 \times 30 = 37.125$

$\underline{P} = P_1 + P_2 = \quad 55.687\ k.$

$R = \dfrac{3 \times 55.687}{2} = 83.53\ k$

$M = 83.53 \times 15 - 55.68 \times 7.5 = 835.35\ Fk.$

Required $S_x = \dfrac{835.35 \times 12}{22} = 455.64\ in^3$

Take W 36 x 150 $\quad S_x = 504.\ in^3$

$\dfrac{150}{45} = 3.33\ \#/\square'$

From page 11 $\quad$ weight of Joist 60' $\quad$ 7.97 $\Big\}$ 11.30 #/
$\quad$ " " Girder $\quad$ 3.33

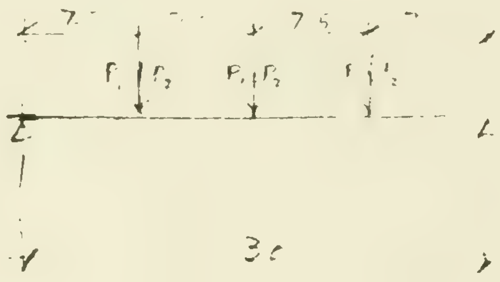
(104)

Girder for support of Joists with 90' span
" " 60' " "

$P_1 = 7.5 \times 165 \times 30 = 37.125^k$

$P_2 = 7.5 \times 165 \times 45 = 55.687$

$P = P_1 + P_2 = 92.812$ Kips



$R = \frac{3 \times P}{2} = 139.22$ K.

$M = 139.22 \times 15 - 92.812 \times 7.5 = 1392.21$ F.K.

Required $S_x = \frac{1392.2 \times 12}{22} = 759.38\ in^3$

take a 36 x 230 $S_x = 837\ in^3$

$\frac{230}{150} = 1.53$

From page 12 weight of girder with 90 = 21.49 17.65

" Girder 1.53

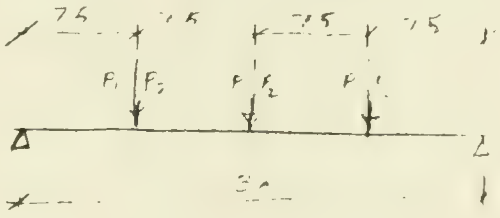
105

Girder for Support of Joist with 90 span one side
Beam " 30 " other side

$P_1 = 75 \times 16.5 \times 15 = 18.562$ Kips

$P_2 = 75 \times 16.5 \times 45 = 55.687$ "

$P = P_1 + P_2 = 74.249$ Kips



$R = \frac{3P}{2} = 111.375$

$M = 111.375 \times 15 - 74.25 \times 7.5 = 1113.75$ F.k.

Required $S_x = \frac{1113.75 \times 12}{22} = 607.5 \text{ in}^3$

Take a 36×182 $S_x = 623$

$\frac{182}{60} = 3.03$ #/sf

See page 12 weight of Joist with 90 = 17.60 21.49 24.5

" " Girder 3.03

106

Load on Cols.

		size
30 × 30 × 150	= 135 + from Top level 67.5 = 202.5	w12 × 53
30 × 45 × 150	= 202.5 + " " " 101.25 = 303.75	w12 × 65
30 × 60 × 150	= 270 + " " 135 = 405	w12 × 87
30 × 75 × 150	= 337.5 + " " " 168.75 = 506.25	w12 × 106

weight of Col per s.f. $= \frac{53}{900} = 0.05$ P/s.f. × 17.5 =

" " " " " $= \frac{65}{1350} = 0.048$ " × 17.5 =

" " " " " $= \frac{87}{1800} = 0.048$ " × 17.5 =

" " " " " $= \frac{106}{2250} = 0.047$ " × 17.5 =

Total weight of steel for Third floor per s.f.

span	Beam	Girder	Col.	Total weight	ref.
30×30	5.33	3.87	0.9	10.10 P/sf	Page 7 8
60×30	7.97	3.33	0.9	12.20 "	" 11 "
90×30	21.49 17.68	1.53	0.9	23.92 19.26 "	" 12,15 16

average

$$\frac{8.5 \times 10.10 + 2 \times 12.20}{10.5} = 10.50$$

$$\frac{7.5 \times 10.10 + 3 \times \overset{23.92}{\cancel{19.26}}}{10.5} = 14.04$$

Average = 12.27 P/sf

Structural Calculations

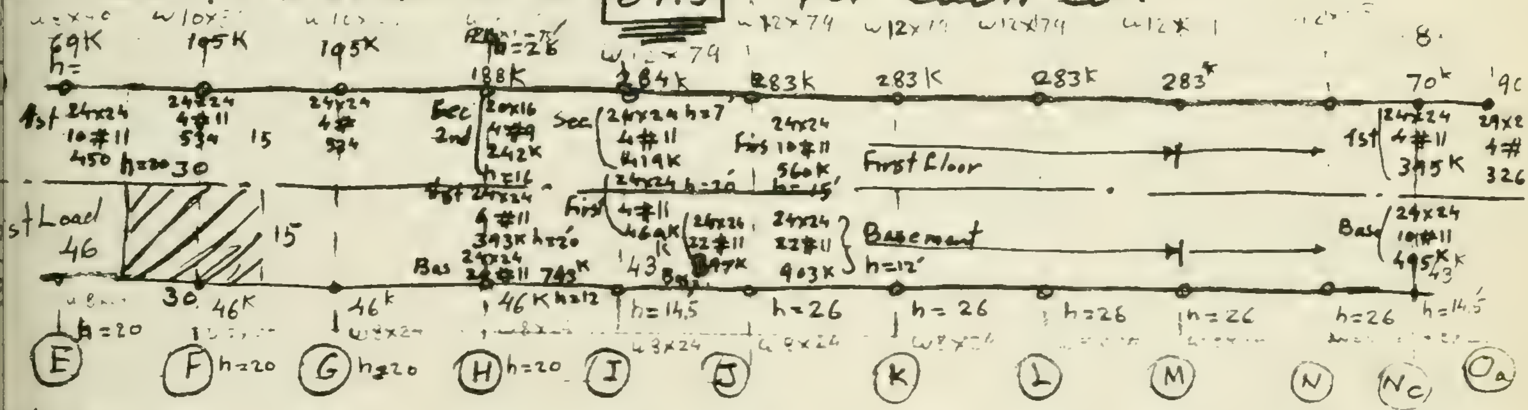
Mechanical Area Along Dalton Street

Weidlinger Associates

Mechanical Room on Top of roof between Axis 8a to 10

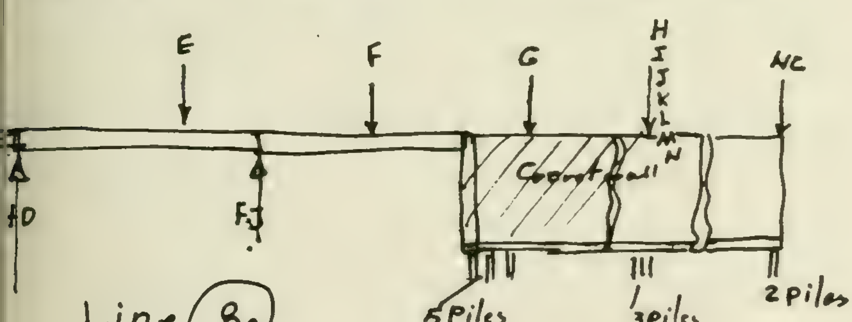
perexiously Load 150 p/sf.

additional L 15 × 30 × 150 = $\boxed{67.5}^K$ for each Col.

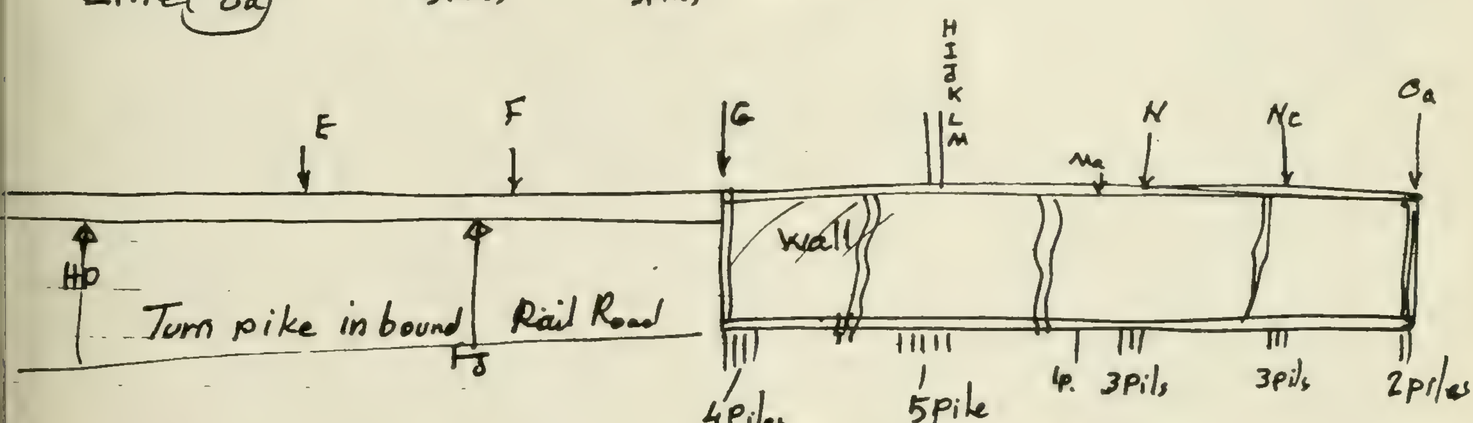


	E	F	G	H	I	J	K	L	M	N	Nc
ext	h=20	h=20	h=20	h=20	h=20	h=20	h=20	h=20	h=20	h=20	24x24
	24x24	24x24	24x24	24x24	24x24	24x24	24x24	24x24	24x24	24x24	8#11
	4#11	4#11	8#11	8#11	8#11	8#11	8#11	8#11	8#11	8#11	h=20
part	534K	534K	428K	428K	428K	211K	211K	211K	211K	211K	395K
	h=20'	h=20'	h=20	h=20	h=20	h=20	h=20	h=20	h=20	h=20	h=20
part			587K	587K	587K	560K	560K	560K	560K	560K	286 ?

Note 5 ?



Line 8a



Line 10

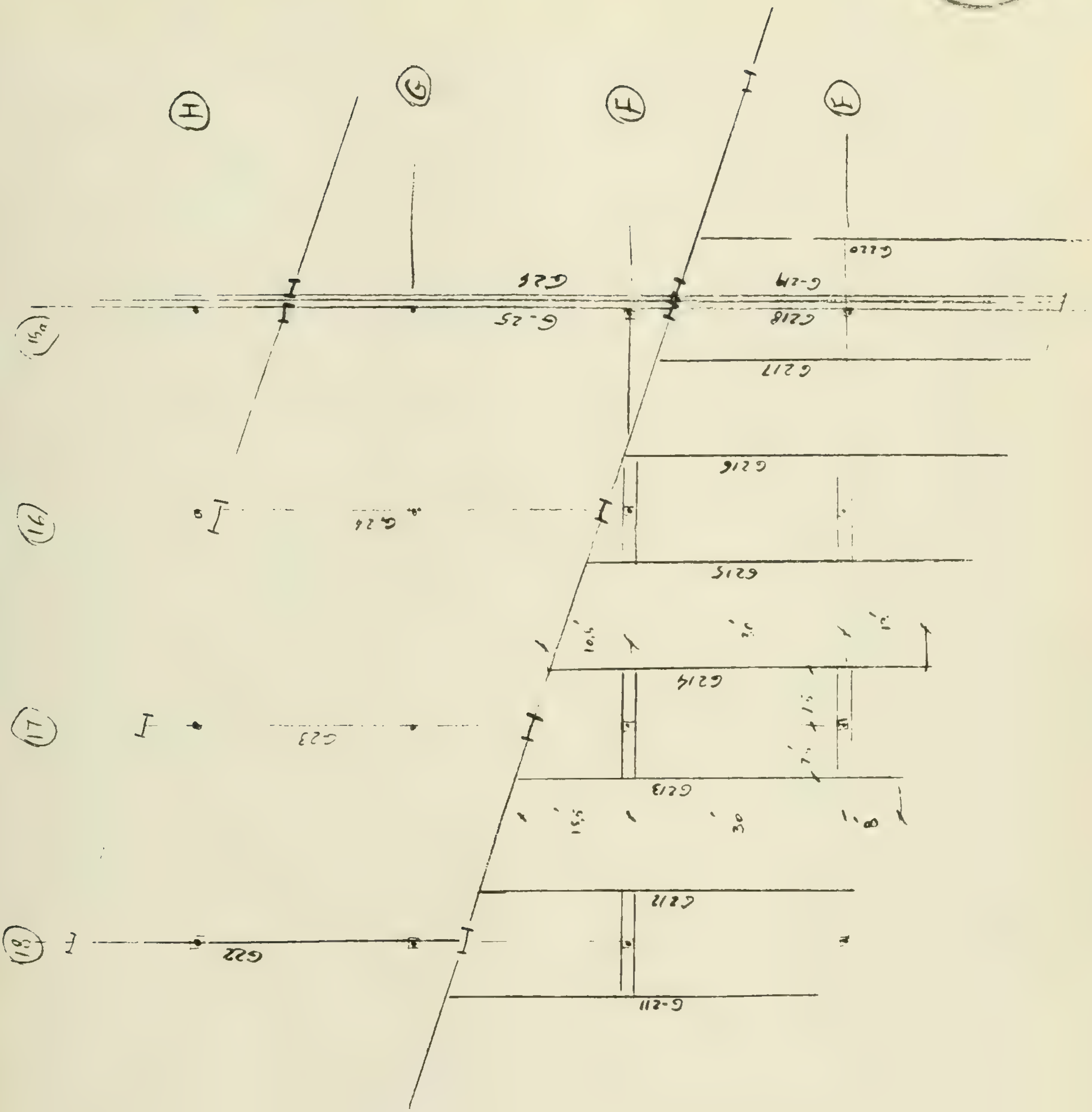
line (8a)

Col	Second Florr Existing Col		Exist Load.	additinal load	Total L.	Capacity	$\frac{L}{r_1}$	Fa		
E	w8x24	h=20'	46K	67.5K	113.5K	48K	$\frac{20'}{3.18}$	10.67		
F	"	"	46K	67.5	113.5K	48K				
G	"	"	46K	67.5	113.5K	43K				
H	"	"	46K	67.5	113.5K	48K				
I	"	14.5'	43K	67.5	110.5K	84.5K	$\frac{14.5'}{3.18}$			
J	"	26'		67.5		28K				
K	"	"		67.5		28K				
L	"	"		67.5		28K				
M	"	"		67.5		28K				
N	"	"		67.5		28K				
N_c	"	14.5'	43K	67.5	110.5K	84.5K				
O_a	—									

Structural Calculations

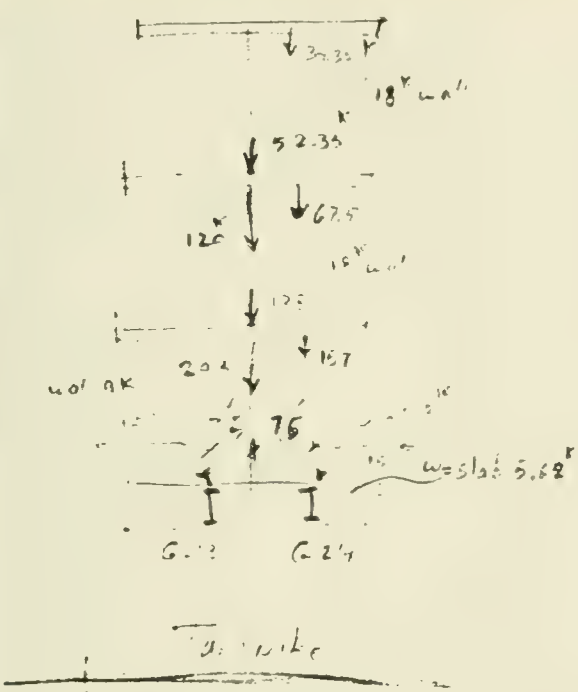
Area Between Line 15a and 18 over Turnpike

Weidlinger Associates

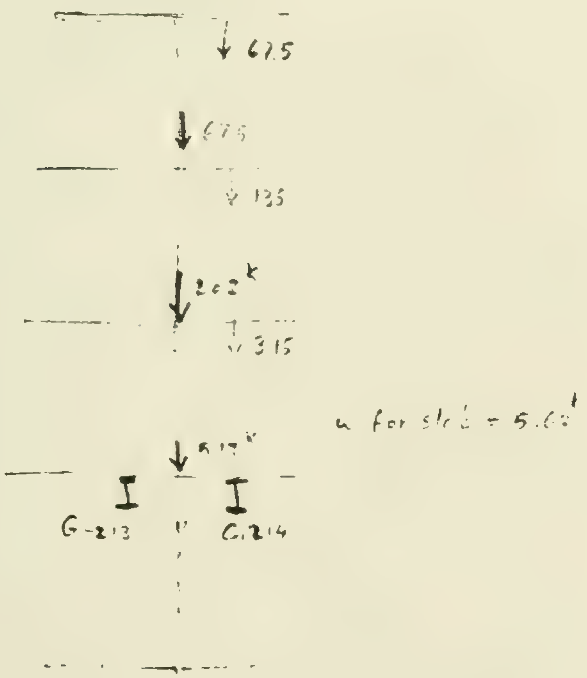

H
G
F
E
15a
16
17
18

Auditorium, ... check of Ext Gir. ...

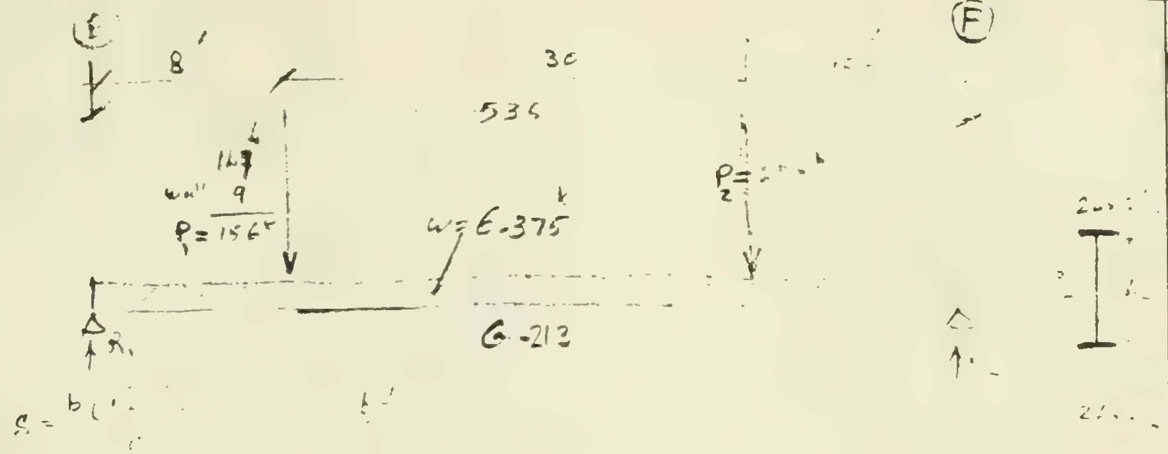
Col. F-17



Col F-17



111



$$R_{p_1} = \frac{156 \times 45.5 + 253 \times 15.5}{53.5} = 207.40^k$$

$$R_{p_2} = 156 + 253 - 207.40 = 206.60^k$$

$$M_p = 207.4 \times 26.75 - 156 \times 15 = 3197.6\ ^{'k}$$

$$M_w = \frac{6.37 \times 53.5^2}{8} = 2279$$

$\Sigma M = M = 5476.60\ ^{'k}$ Required $S = 2987.27$

<u>Existing Girder</u>

$$S = \frac{26(47^3 - 42^3)}{6 \times 47} + \frac{0.75 \times 42^2}{3} = 3182.5\ in^3$$

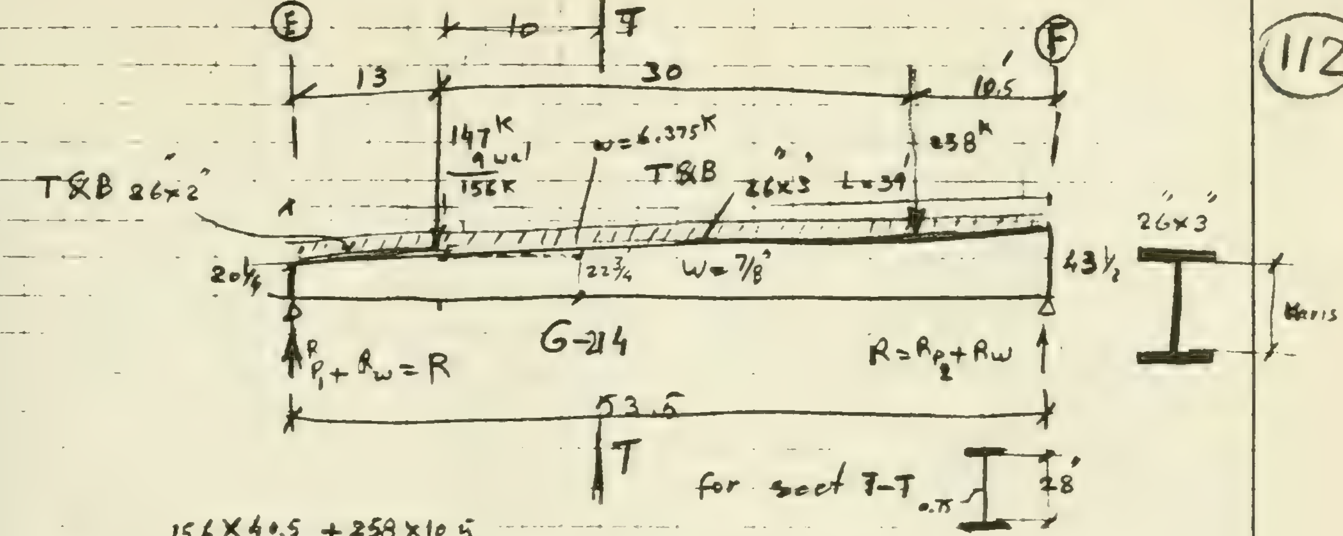
Existing $S = 3182.5$ > Required $S = 2987.27$ ok

Shear = web $A_s = 42 \times 0.75 = 31.5$

$$R_1 = R_{p_1} + R_w = 207.4 + \overset{170.40^k}{\frac{6.37 \times 53.5}{2}} = 377.79^k$$

$$R_2 = R_{p_2} + R_w = 206.6 + 170.40 = 377.-$$ $v_v = \ldots \cong 14^k$

$$f_v = \frac{377.79}{31.5} = 11.99^k < 14.^k$$ ok



$$R_{P_1}=\frac{156\times 40.5+258\times 10.5}{53.6}=168.73\text{ K}$$

$$R_{P_2}=156+258-168.73=245.27\text{ K}$$

$$M_{P_{T-T}}=168.73\times 23-156\times 10=2320\text{ F.K.}$$

$$M_{w_{T-T}}=\frac{6.37\times 23\times 30.5}{2}=2234\text{ F.K}\qquad M_x=\frac{w\times(l-x)}{2}$$

$$M=\Sigma M=2320+2234=4554\text{ F.k.}$$

$$\text{Required } S=\frac{4554\times 12}{22}=2484.0\text{ in}^3$$

Existing Girder: Modulus Sat --- section

$$S=\frac{26''(34^3-28^3)}{6\times 34}+\frac{0.875\times 28^2}{3}=2440.20\text{ in}^3$$

Existing Modulus section $=2440.20\text{ in}^3 \approx$ Required Modulus sect. $=2484.\text{ in}^3$ OK

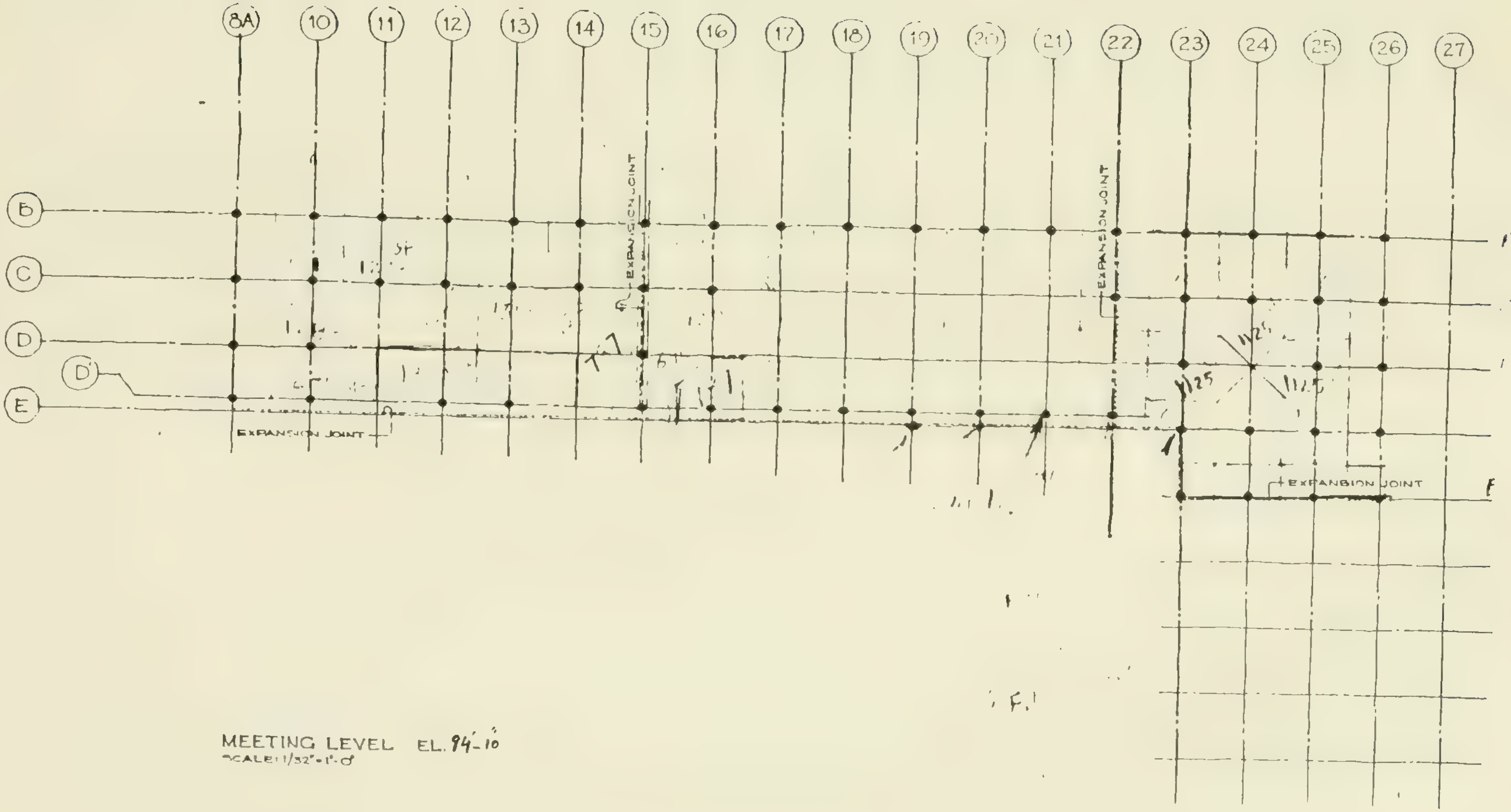
Shear:

$$R_1=168.73+\overbrace{\frac{6.37\times 53.5}{2}}^{170.40}=339.13\text{ K}\qquad \text{web } A_s=0.875\times 20.25=17.72\text{ in}^2$$

$$R_2=245.27+170.4=415.67\text{ K}\qquad \text{web } A_s=0.875\times 43.50=38.06\text{ in}^2$$

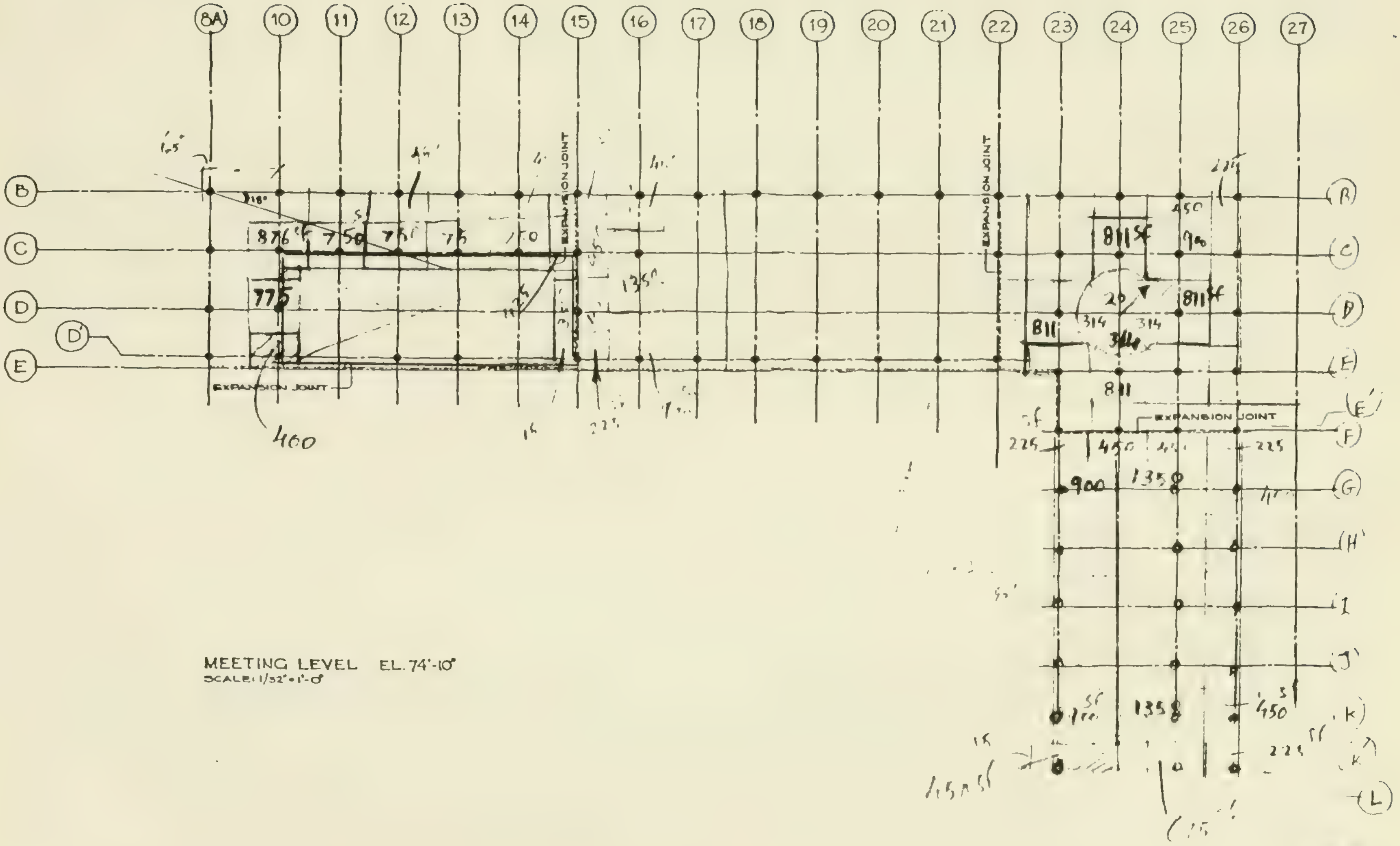
$$\sigma_{v_{R_1}}=\frac{339.13}{17.72}=19.15^K>14.40 \rightarrow \text{to be reinforced.}\qquad v_v=0.40f_y=14.40^K$$

$$\sigma_{v_{R_2}}=\frac{415.67}{38.06}=10.92^k<14.40\ \text{OK}$$



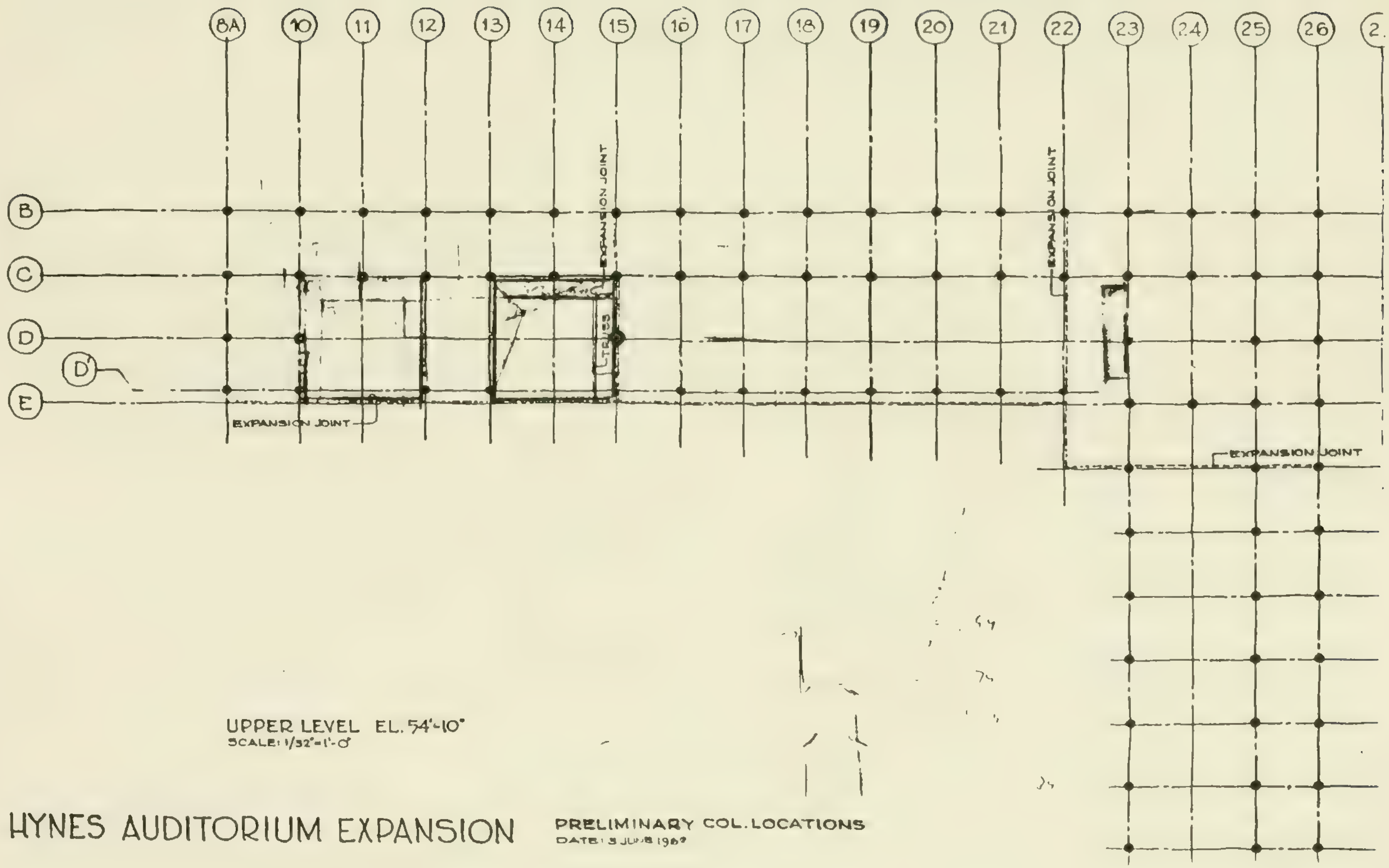
MEETING LEVEL EL. 94'-10"
SCALE: 1/32" = 1'-0"

113



MEETING LEVEL EL. 74'-10"
SCALE: 1/32" = 1'-0"

114

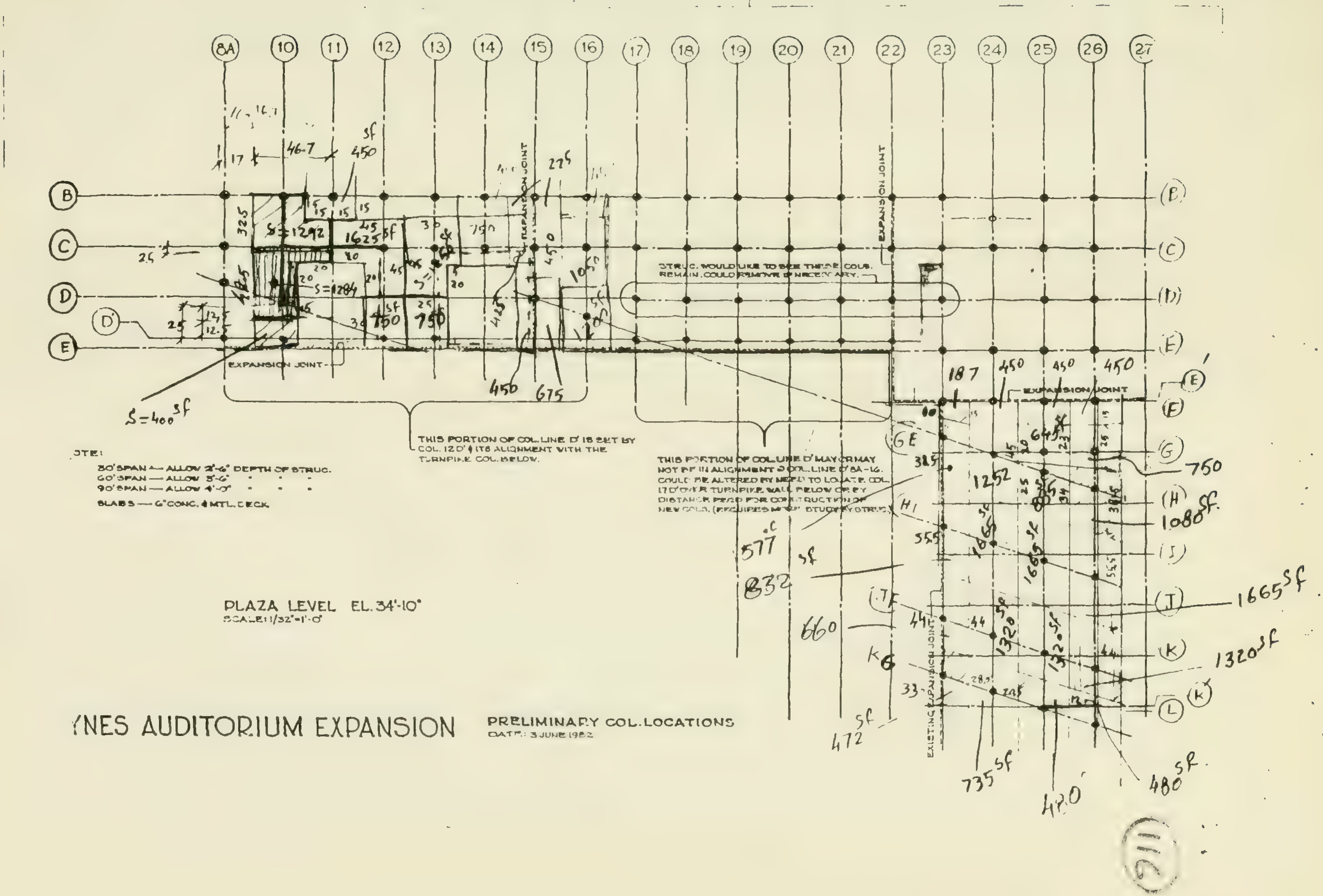


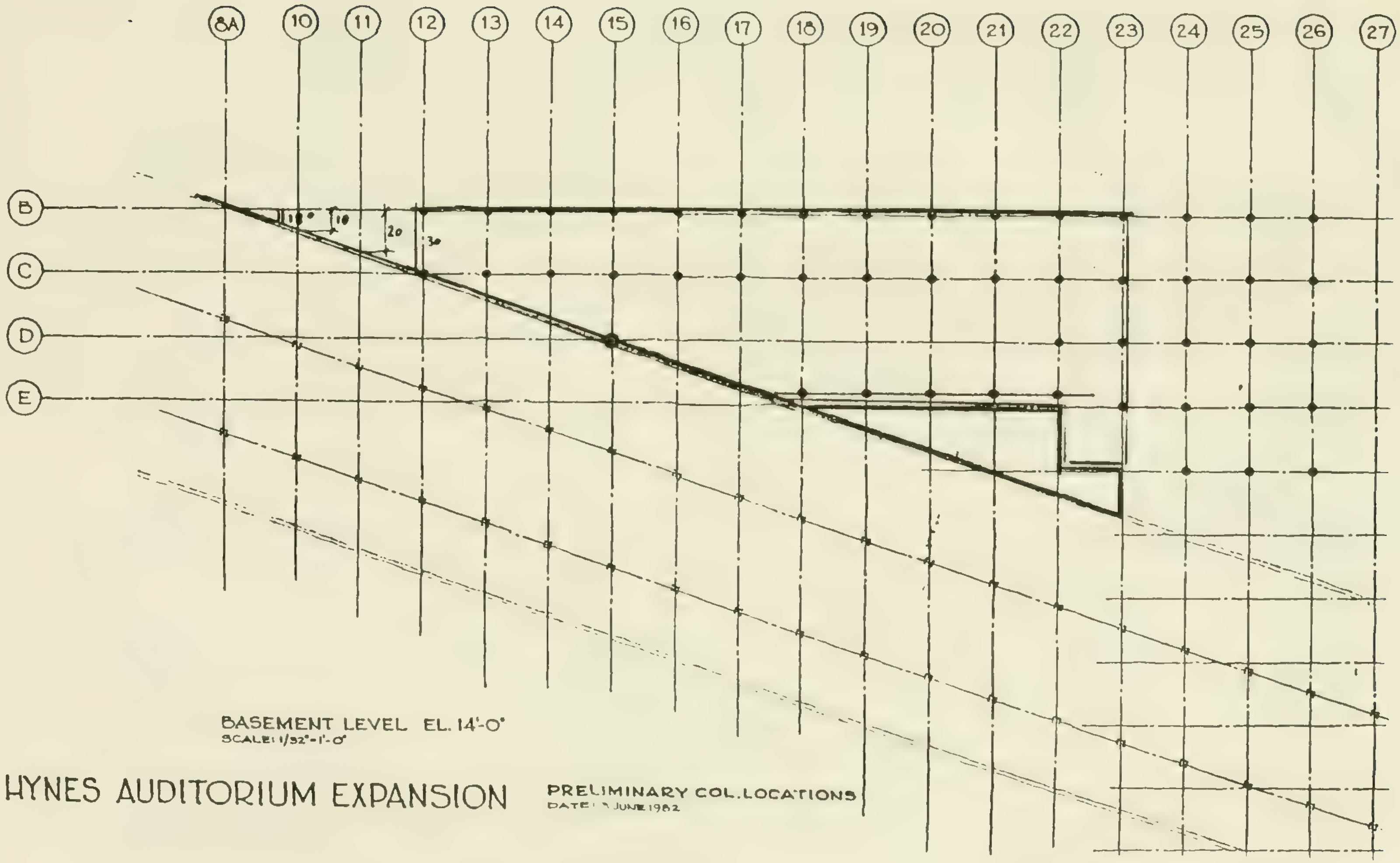
UPPER LEVEL EL. 54'-10"
SCALE: 1/32"=1'-0"

HYNES AUDITORIUM EXPANSION

PRELIMINARY COL. LOCATIONS
DATE: 3 JUNE 196[illegible]

115


YNES AUDITORIUM EXPANSION
PRELIMINARY COL. LOCATIONS
DATE: 3 JUNE 1982
PLAZA LEVEL EL. 34'-10"
SCALE: 1/32"=1'-0"
OTE:
30' SPAN — ALLOW 2'-6" DEPTH OF STRUC.
60' SPAN — ALLOW 3'-6"
90' SPAN — ALLOW 4'-0"
SLABS — 6" CONC. & MTL. DECK
THIS PORTION OF COL. LINE D' IS SET BY COL. 12D' & ITS ALIGNMENT WITH THE TURNPIKE COL. BELOW.
STRUC. WOULD LIKE TO SEE THESE COLS. REMAIN, COULD REMOVE IF NECESSARY.
EXPANSION JOINT
EXISTING EXPANSION JOINT
S=400 sf
750
1080 sf
1665 sf
1320 sf
480 sf
735 sf
472 sf
660
577
832

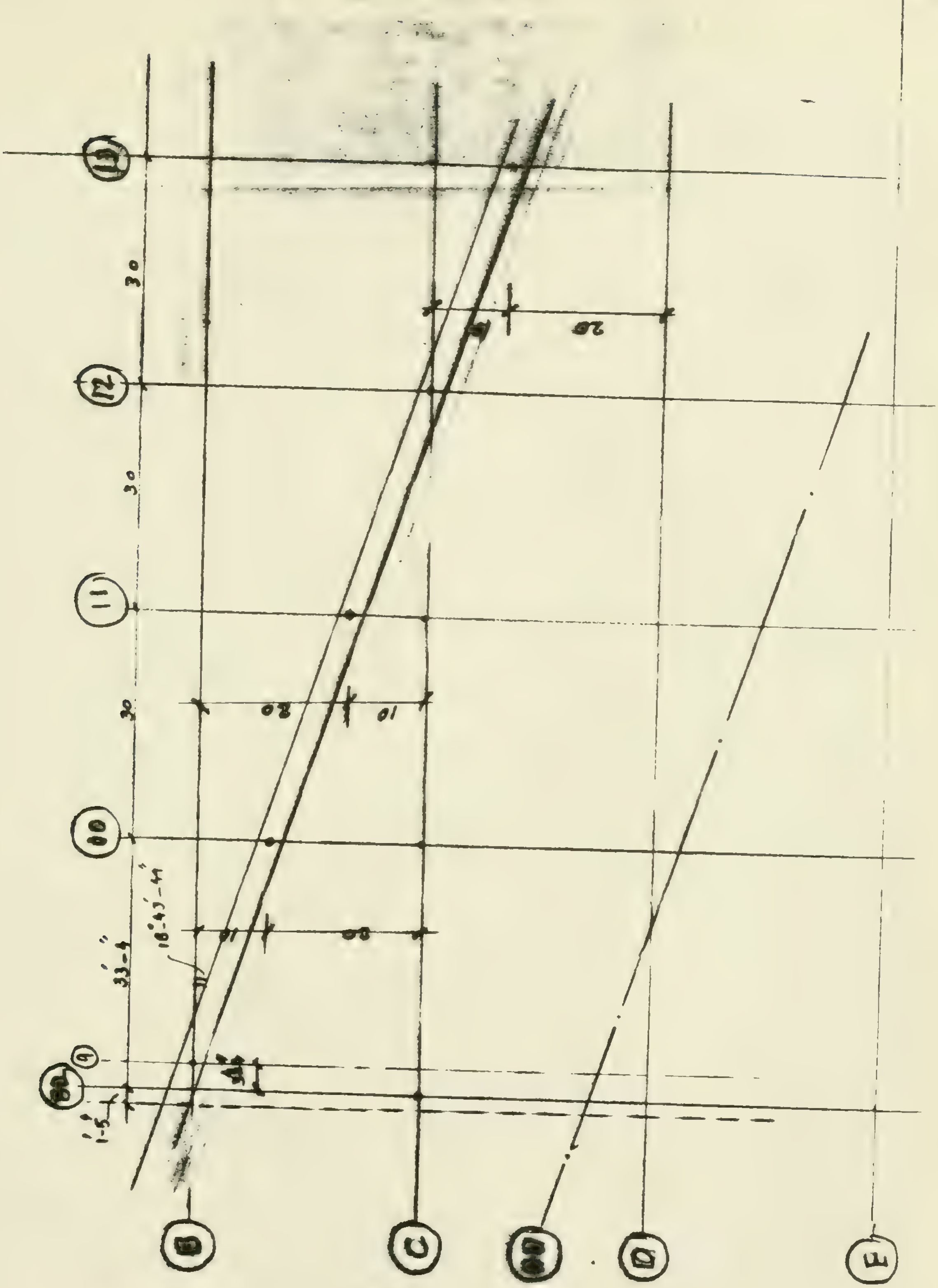


BASEMENT LEVEL EL. 14'-0"
SCALE: 1/32"=1'-0"

HYNES AUDITORIUM EXPANSION

PRELIMINARY COL. LOCATIONS
DATE: 3 JUNE 1982

117



13
12
11
10
9
30
30
30
20
10
20
10
33'-4"
B
C
D
E

Roof = 110 #/□'

Loads: Typ. = 160 #/sf

Side wall 20' hight = 600 #/ft linear

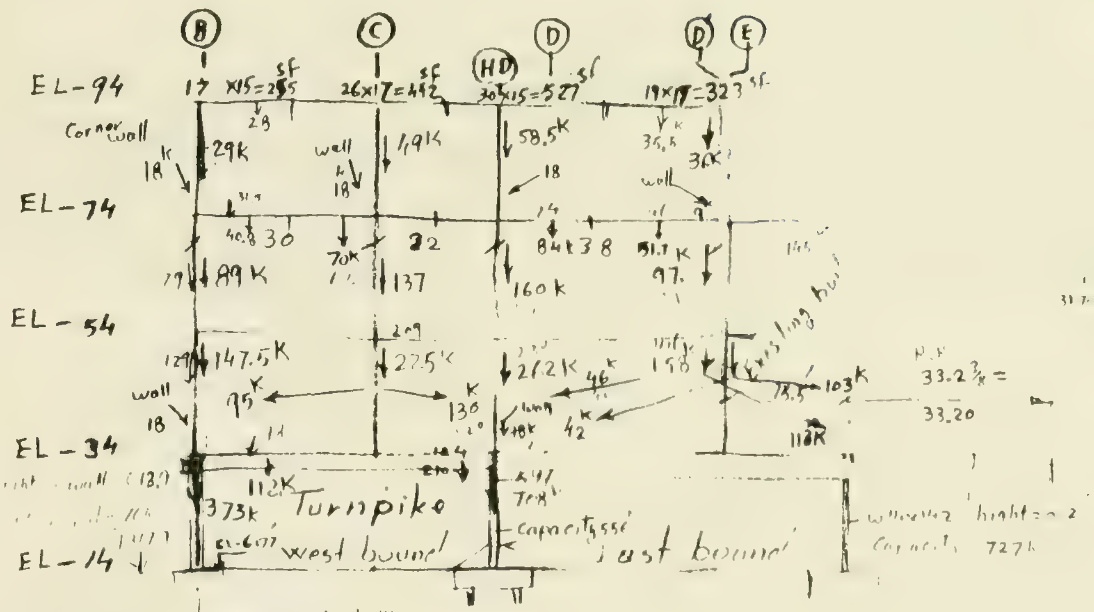
Parapet = 20 #/ft

Groun floor = 250 #/sf

Reduction for area more than 150 sf only %20 of live load

$\frac{33.4 + ...}{2}$ 31.7'

Line 8a



33.2 ³⁄₈ = 33.20

capacity 727k

$\frac{1377}{2} + 373 = 1061^k$

Capacity = 2400 kip

2,17

10x2.17x30x145 = 93,395

2x0.67x6x30x145 = 24,120

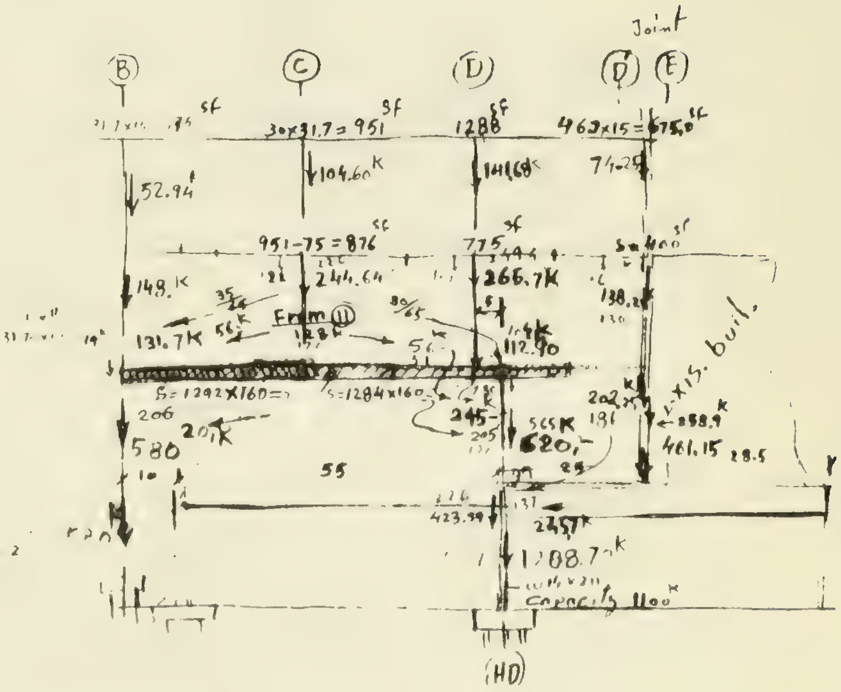
Total 117,515

12.88.7

2313k

Piles Capacity

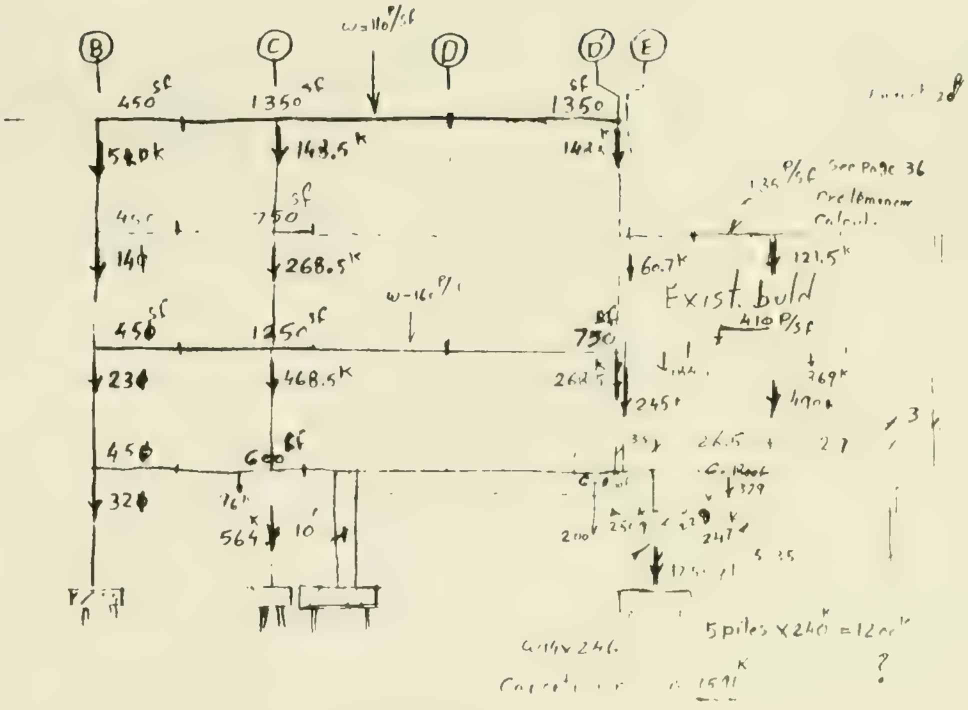
Line 10



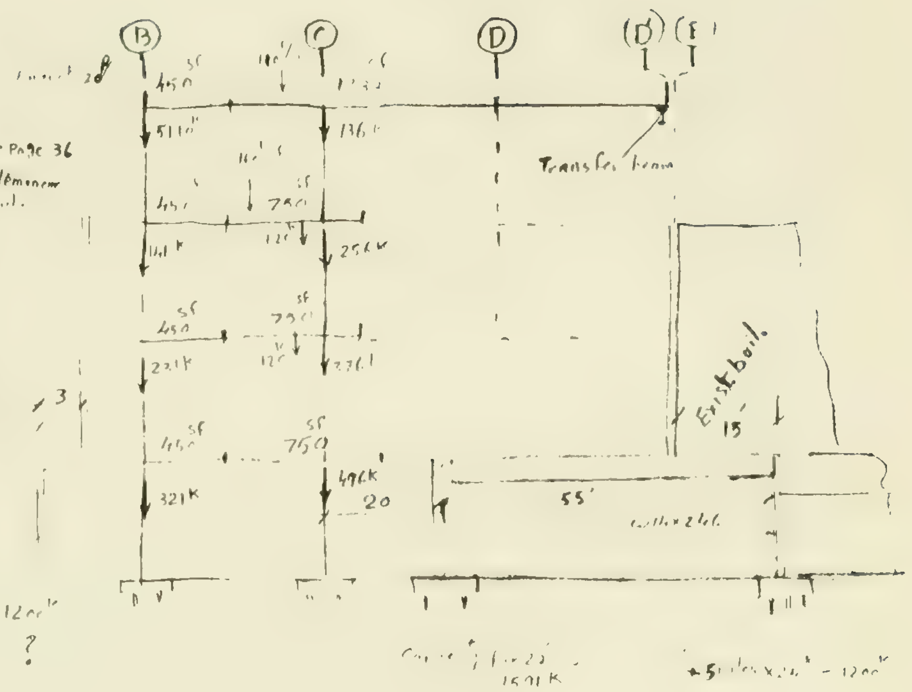
to b rienforced ?

after reduction

Line 13

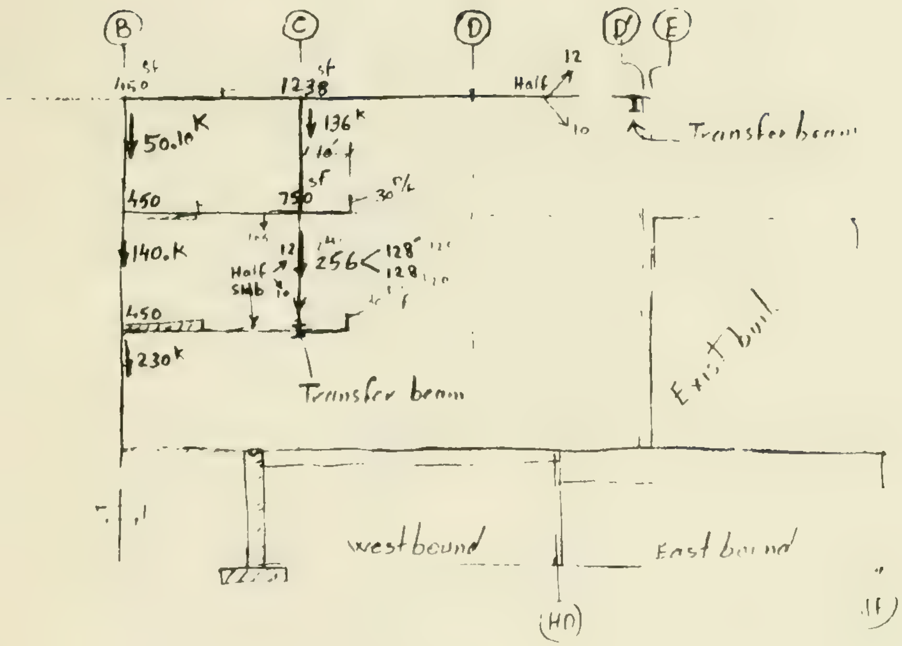


Line 14



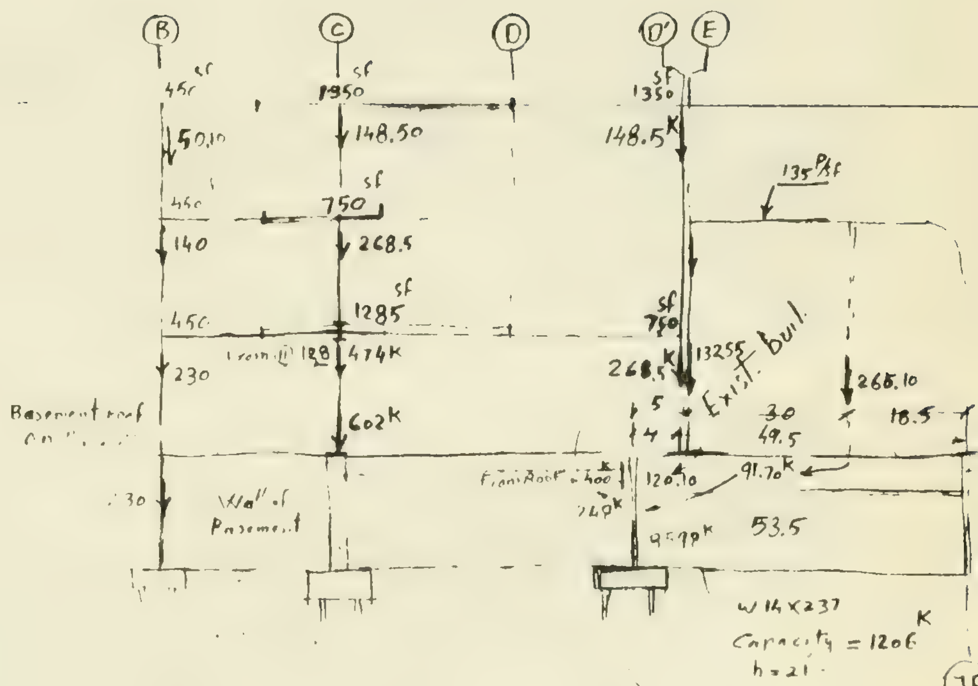
1255.9
Turnpike wall 100
1356K
> 1200

Line 11



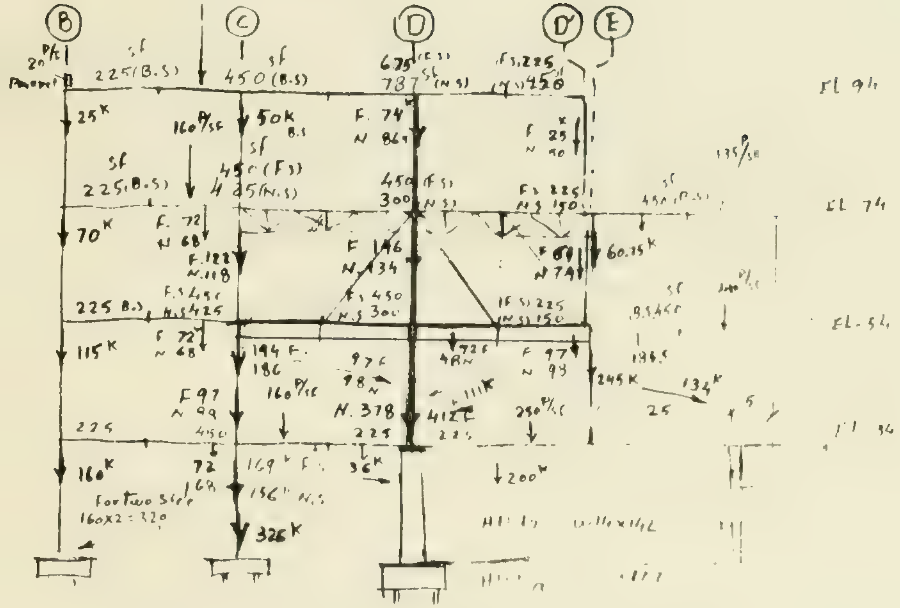
OK

Line 12



121

Line 15
(Expansion Joint)

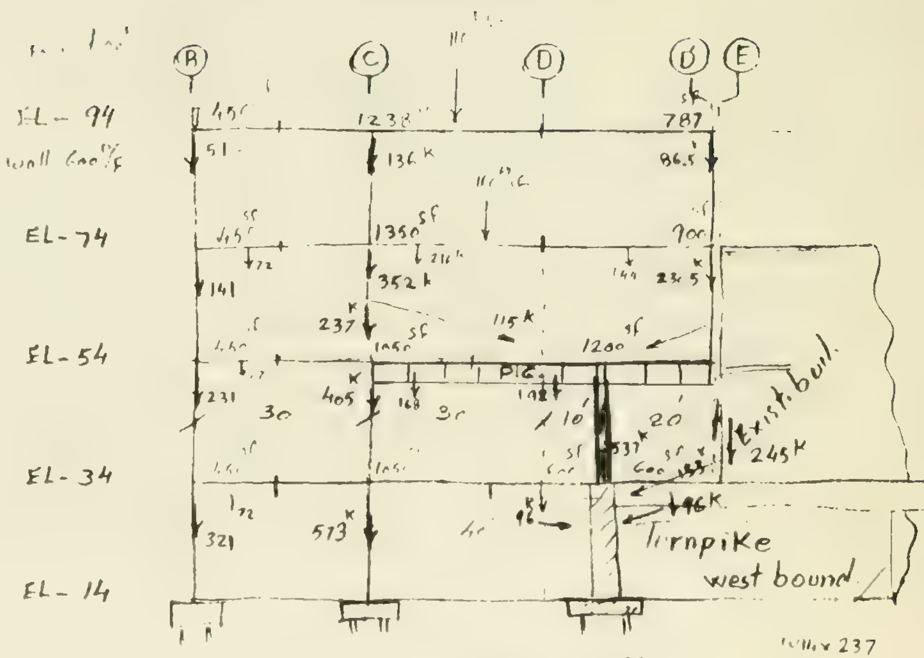


Total Load on up the wall. 378
36
412
200
1037
weight of concrete 612 K

OK

Capacity of the piles

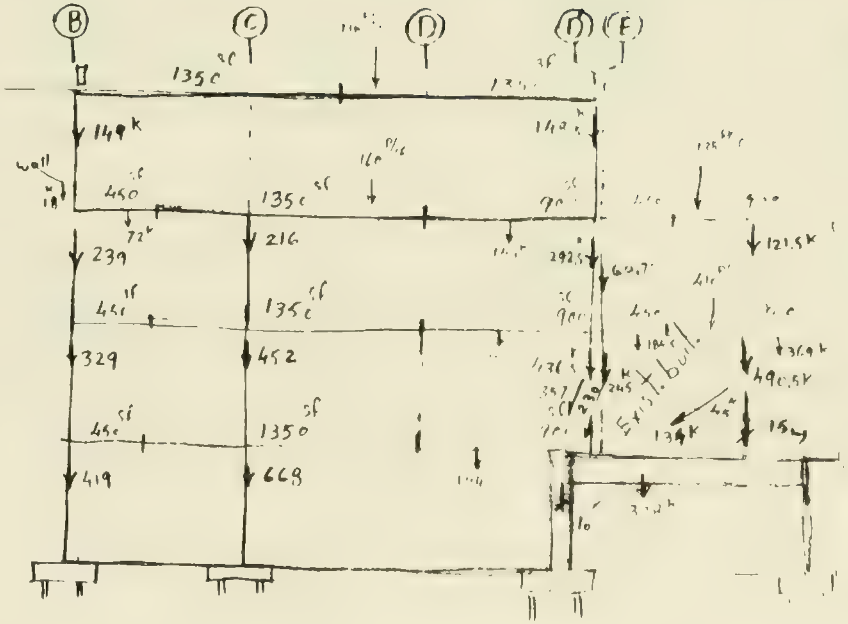
Line 6

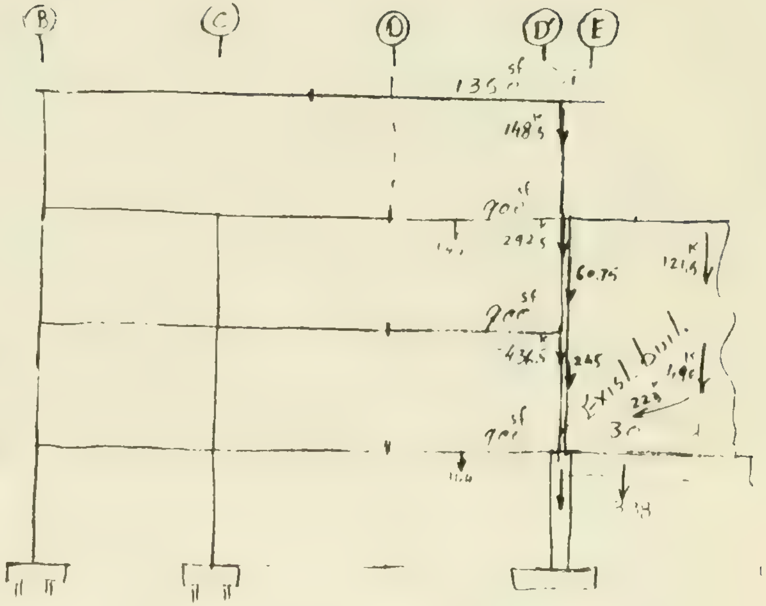


Total load 96
96
537
729
(only) weight of conc. alud 612
1341 } 1594 K
From Exist Build. 153 }
OK
Footing capacity

Line 1;

Line 1e



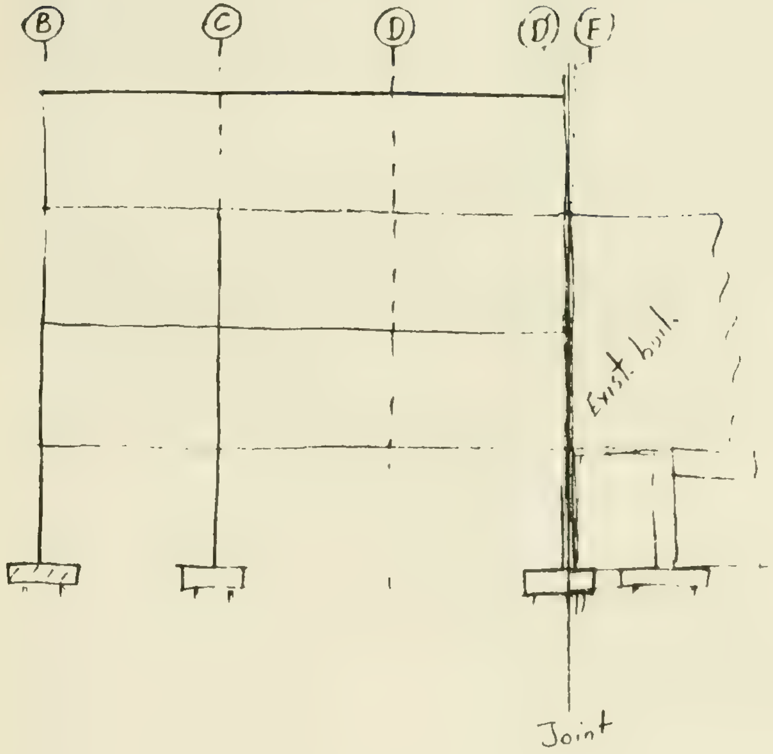


Total load 357
230
114
338
134
1303
weight of conc. of wall 612
19[illegible]5

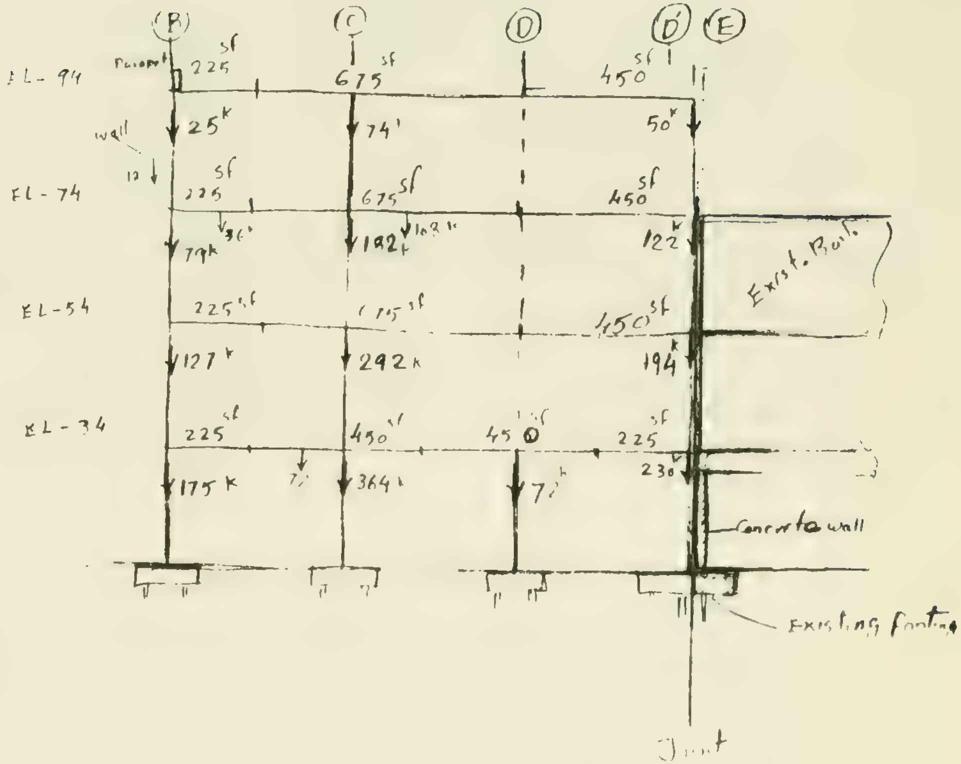
[illegible] capacity of piles
10×240 = [illegible]k

Total load 436.5k
245
144
223
338
1386.5
weight of wall 612
1998.5 < 2400

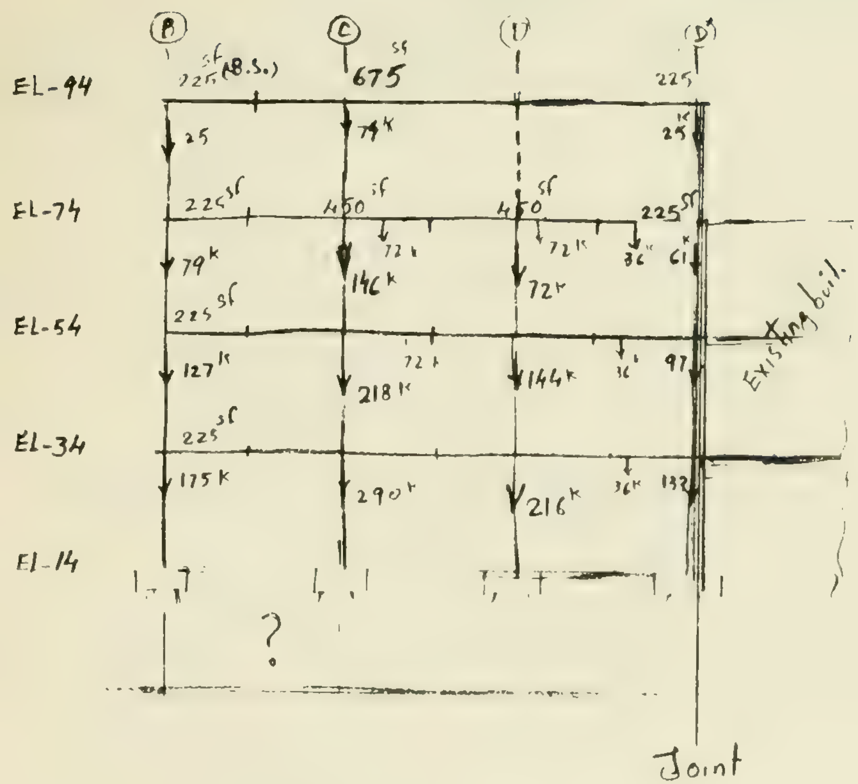
Line 19

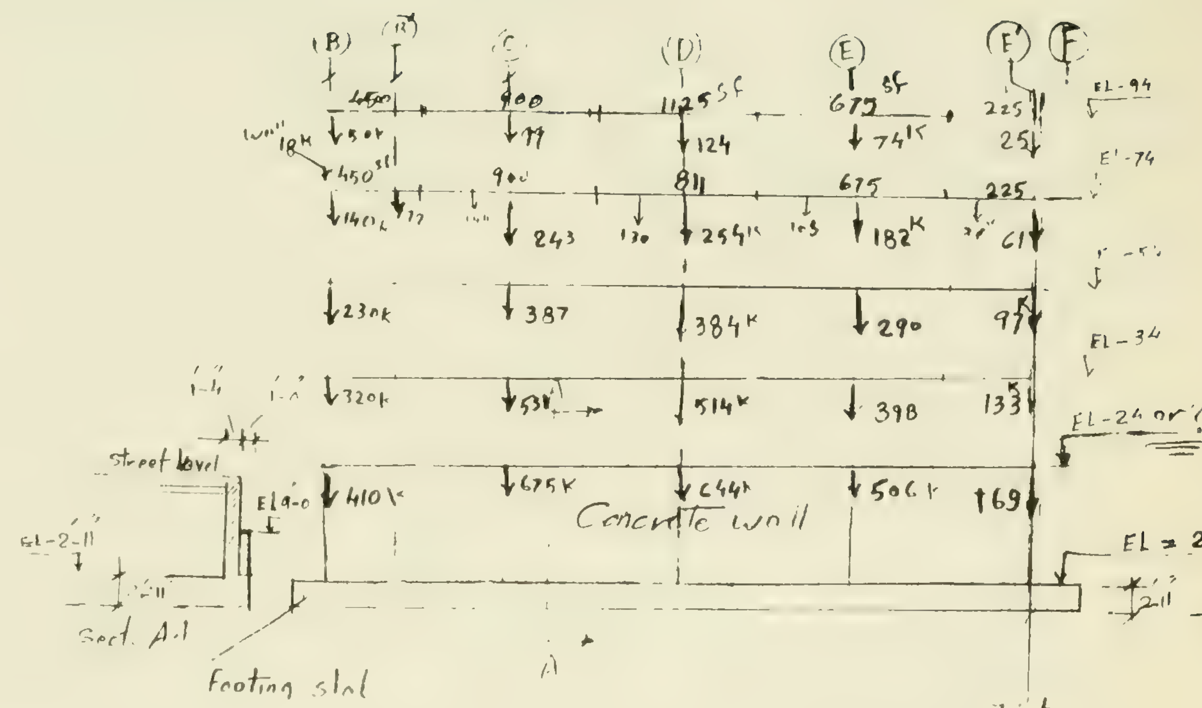


Line 22



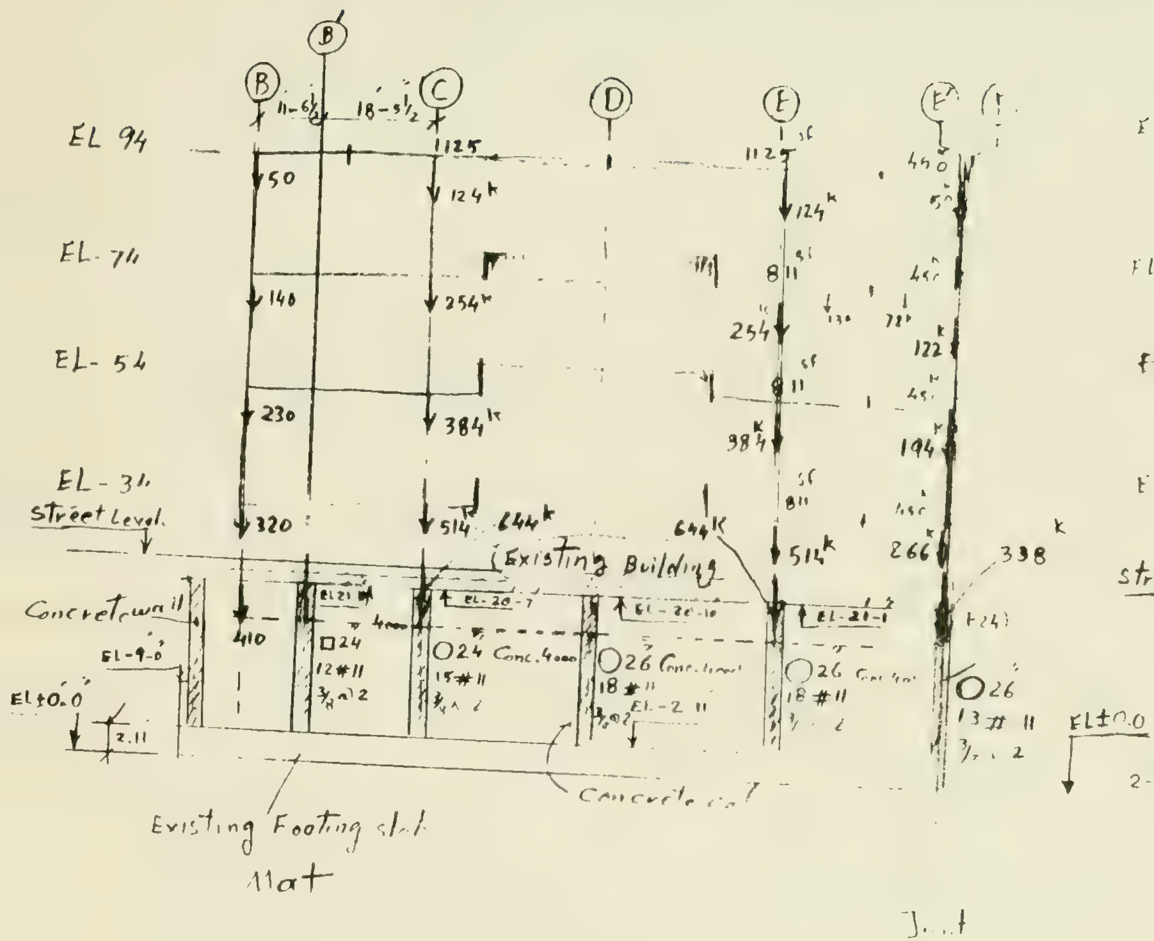
Line 2.2'
Expan...
After Joint



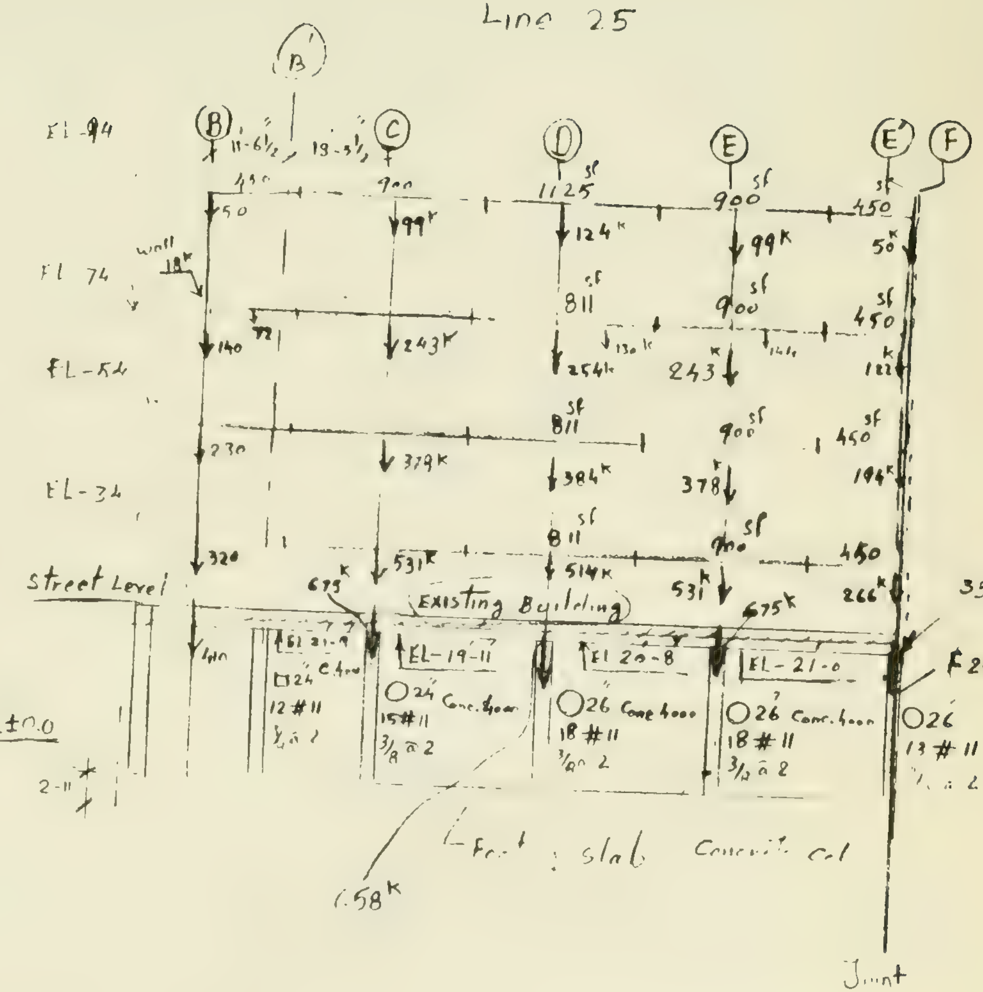


what will hapen
between the Elevation
EL-24'
EL-2-11

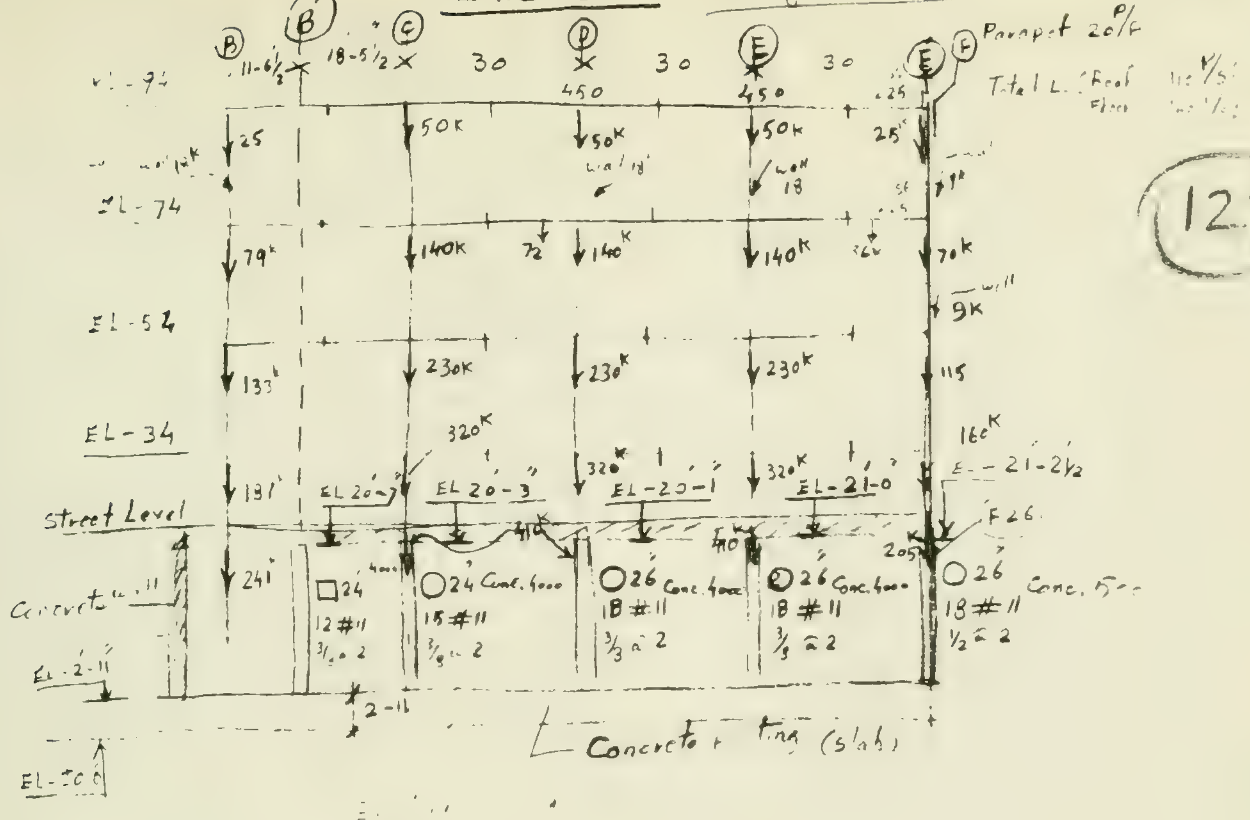
Line .:

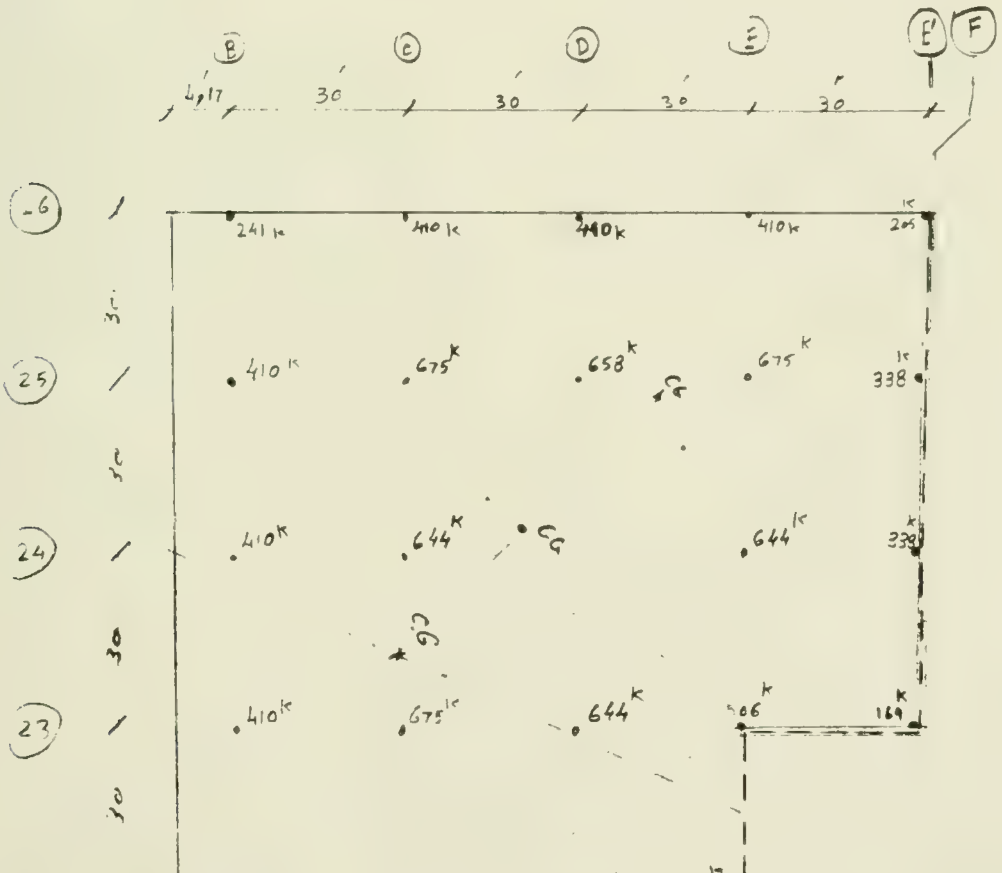

EL 94
EL-74
EL-54
EL-34
Street Level
Concrete wall
EL-9-0
EL±0.0
Existing Building
Existing Footing slab
Mat
Concrete col
Joint

Line 25


EL-94
EL 74
EL-54
EL-34
Street Level
Existing Building
EL-19-11
EL 20-8
EL-21-0
EL±0.0
Foot slab
Concrete col
Joint

Line 26 Faicing Wall 600#/F





From Col. Total load = 9911K

Tatal area = 14000

weight of Mat = 14000 × 2.92 × 145 = 5927K

from El-3' to 14' ?

$$\text{stress of soil} = \frac{9911 + 5927}{14000} = 1.42\ ^{K}/_{sf}.$$

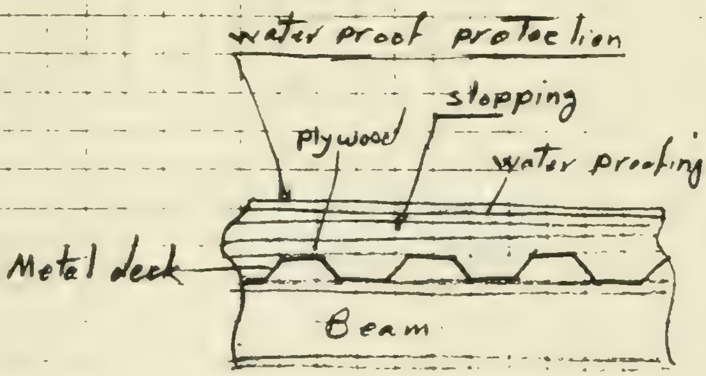
Structural Calculations

Area Between Line 18 and 22a over Turnpike

Weidlinger Associates

Loads.

a) Roof.

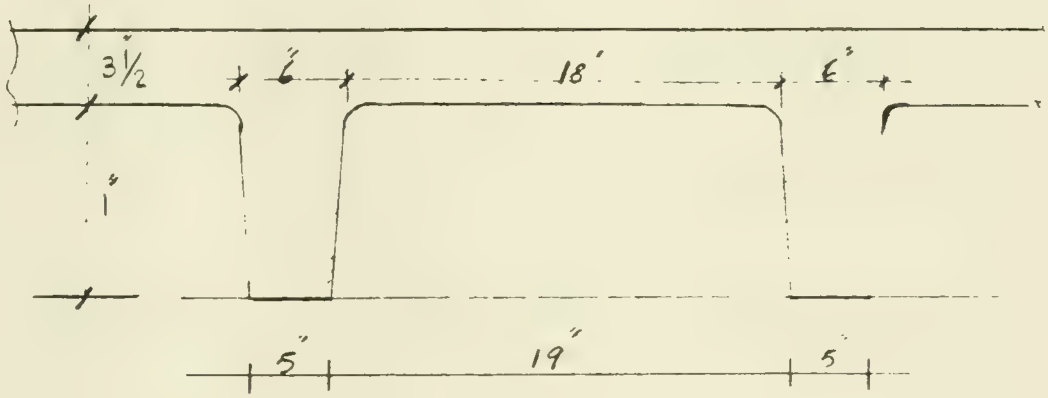


D.L -	All Materials of roofing	25	P.S.F
	Beams	5	"
	Mechanical and others (ceiling)	15	"
L.L	Snow	30	"
	Total load.	75	P.S.F.

b) Third floor:

D.L.	slab 4" concrete + deck	50	P.sf
	Beams	5.-	"
	Mechanical and others	15	"
L.L 100 P/sf Reduced 20% for area > 150 sf		80	
	Total load	150	P.sf.

Existing Floor on second level.



weight of Two ways Ribd slab: (for a part $2' \times 2' = 4^{sf}$

$$slab = 2' \times 2' \times 0.2917 \times 150^{lb} = 175 \text{ lb.}$$

$$Ribs = \frac{\frac{2.5+3}{2} \times (19+25) \times 1 \times 4}{144} \times 150 = 246.35$$

$$421.35$$

$$\frac{421.35}{4} = 105.30 \text{ P.S.F.}$$

add 10% for beams $= 105.30 \times 1.10 = 116$ P/S.F

So:

D.L =	slab	116 P/sf
	Finishing Floor	15 "
		131 P/sf
L.L 250 P/sf area > 150 sf Reduced 20%		200 " (For column design)
	Mechanical and other	15
		346 P/s.f

take 350 P/s.f. for Col. Calcul.

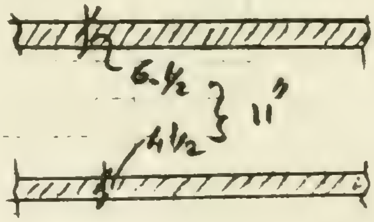
400 P/sf for Beams Calcul.

Auditorium, Boston, MA. «Addition»

Existing slabs at Ground Floor. (Roof of turnpike tunel)

(131)

Reinforced concrete slabs.



D.L. weight of slabs $= \frac{11 \times 145}{12} = 133$ lb/s.f

beams and Girders & other 17

Finishing floor 25

L.L 250 P/sf Reduced 20% for area > 150 sf 200 (for col. design only)

375

take 375 P/sf. For Col. calcul.

425 P/sf For Beams calcul.

walls.

Exterior wall 600 P/f linear

Parapet 20 P/f linear

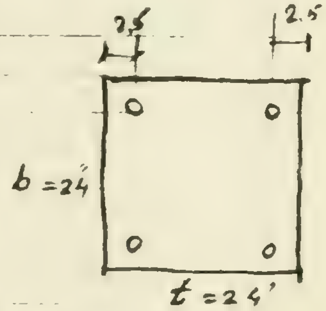
Auditorium, Existing Cols. « Addition »

Capacity of Concrete Cols.

Section Type 1 & 2 4-# 11 $24'' \times 24''$ $\# = 1.56 \text{ in}^2$

Concrete 5000 psi or 5. ksi

steel rod 50000 or 50 ksi



$$g = \frac{24 - 2 \times 2.5}{24} = 0.79 \rightarrow 0.8$$

Page 59 chart No-19 (uniaxial)

From Ultimate strenght Design Handbook volume 2 Cols.

excentricity as code Mini- 0.1 Assume: $e = 4''$

$$\frac{e}{t} = \frac{4}{24} = 0.17 \qquad \rho_t = \frac{A_s}{A_c} = \frac{4 \times 1.56}{24 \times 24} = 0.011$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{50}{0.85 \times 5} = 11.76 \qquad k = 0.44$$

$$k = \frac{P_U}{f_c' \times b t} \qquad P_U = 0.44 \times 5 \times 24 \times 24 = \underline{1267.2 \text{ kips}}$$

Auditorium, Existing Cols. (Addition)

133

Capacity of Concrete Cols.

Section (2) 6 # 11 $24'' \times 24'' = b \times t$

$f'_c = 5$ ksi

$F_y = 50$ ksi $g = 0.8$

excentricity as code Mini 0.1 Assume $e = 4''$

$\frac{e}{t} = \frac{4}{24} = 0.17$ $\rho_t = \frac{A_s}{A_c} = \frac{6 \times 1.56}{24 \times 24} = 0.016$

chart No. 19 $k = 0.45$

$k = \frac{P_u}{f'_c b t}$

$P_u = 0.45 \times 5 \times 24 \times 24 = \underline{1296\ k}$

Capacity of Concrete Cols.

Section Type 3

8 # 11 $b = 24''$

$t = 24''$

$f_c' = 5$ ksi

$f_y' = 50$ ksi $g = 0.8$

Assume $e = 4''$

$\frac{e}{t} = \frac{4}{24} = 0.17$ $\rho_t = \frac{As}{Ac} = \frac{12.48}{576} = 0.22$

From chart 19 $k = 0.49$

$k = \frac{P_u}{f_c' b t}$

$P_u = 0.49 \times 5 \times 576 = \underline{1411.2}$ ksi

Capacity of Concrete Columns.

Section Type 9

4 #6 $b = 12''$

$t = 12''$

$f'_c = 5$ ksi

#6 $= 0.44\ in^2$

$f_y = 50$ ksi

$g = \frac{12-5}{12} = 0.58 \longrightarrow g = 0.6$

Excentricity as Code (Mini) 0.1 Assume $e = 2.4''$

$\frac{e}{t} = \frac{2.4}{12} = 0.2$

$\rho_t = \frac{4 \times 0.44}{12 \times 12} = \frac{1.76}{144} = 0.012$

$K = 0.40$

$P_U = 0.40 \times 5 \times 12 \times 12 = 288$ kips.

Capacity of Concrete columns.

Section Type (10) $4 \# 11$

$b = 24 \quad t = 29$

$g = \frac{29-5}{29} = 0.83 \longrightarrow g = 0.8$

Assume excentricity $e = 4''$

$\frac{e}{t} = \frac{4}{29} = 0.03$

$\rho t = \frac{4 \times 1.56}{696} = 0.008965 \longrightarrow 0.009$

$K = 0.58$

$P_u = 0.58 \times 5 \times 24 \times 29 = \underline{2018}$ kips

Capacity of Concrete Columns

Section Type 18

$F_c' = 5$ ksi

$F_y = 50$ ksi

16 # 11 $b = t = 24''$

excentricity (Mini) as code is 0.1t Assume $e = 4''$

$$\frac{e}{t} = \frac{4}{24} = 0.17$$

$$\rho_t = \frac{A_s}{A_c} = \frac{16 \times 1.56}{576} = 0.02979 \longrightarrow 0.03$$

$k = 0.52$

$P_u = 0.52 \times 5 \times 24 \times 24 = \underline{1497}$

Capacity of Concrete columns.

Section Type 20 $20 \# 11$

$b = t = 24''$

$f'_c = 5$ ksi

$f_y = 50$ ksi

Excentricity (Mini.) as code alt Assume $e = 4''$

$$\frac{e}{t} = \frac{4}{24} = 0.17$$

$$f_t = \frac{A_s}{A_c} = \frac{20 \times 1.56}{24 \times 24} = 0.05416$$

$K = 0.62$

$P_u = 0.62 \times 5 \times 24 \times 24 = \underline{1785.6}$ kips

Capacity of Concrete Columns.

Section Type 30 $14 \# 11$

$b = t = 24''$

$f'_c = 5$ ksi

$f_y = 50$ ksi

Eccentricity mini. as code is 0.1t Assume $e = 4''$

$$\frac{e}{t} = \frac{4}{24} = 0.17$$

$$\rho_t = \frac{A_s}{A_c} = \frac{14 \times 1.56}{24 \times 24} = 0.0379 \rightarrow 0.038$$

$K = 0.56$

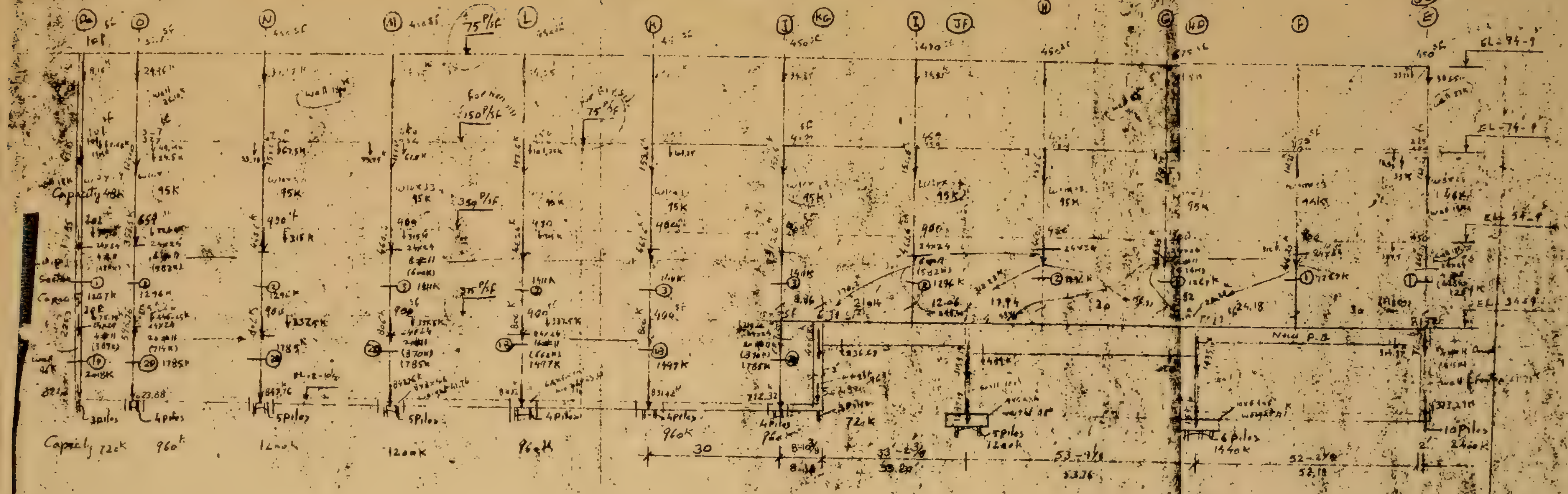
$$P_u = 0.56 \times 5 \times 24 \times 24 = 1612.8 \text{ kips}$$

Structural Calculations

Cross Sections Between Lines 18 and 22a

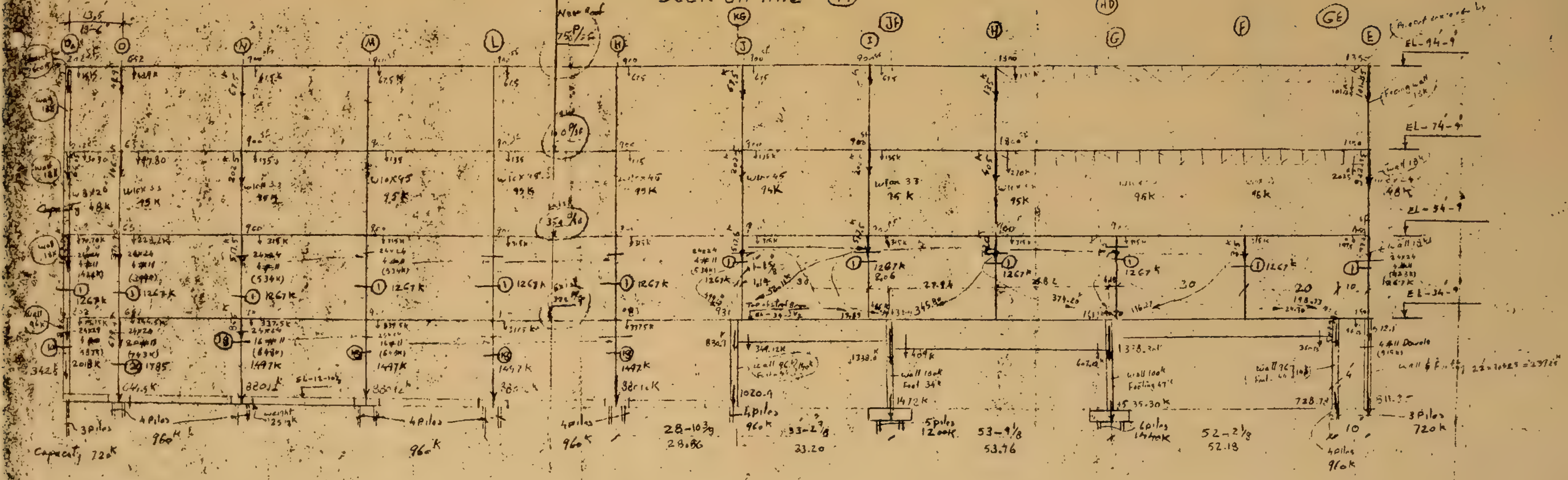
Weidlinger Associates

Sect. on line (18)

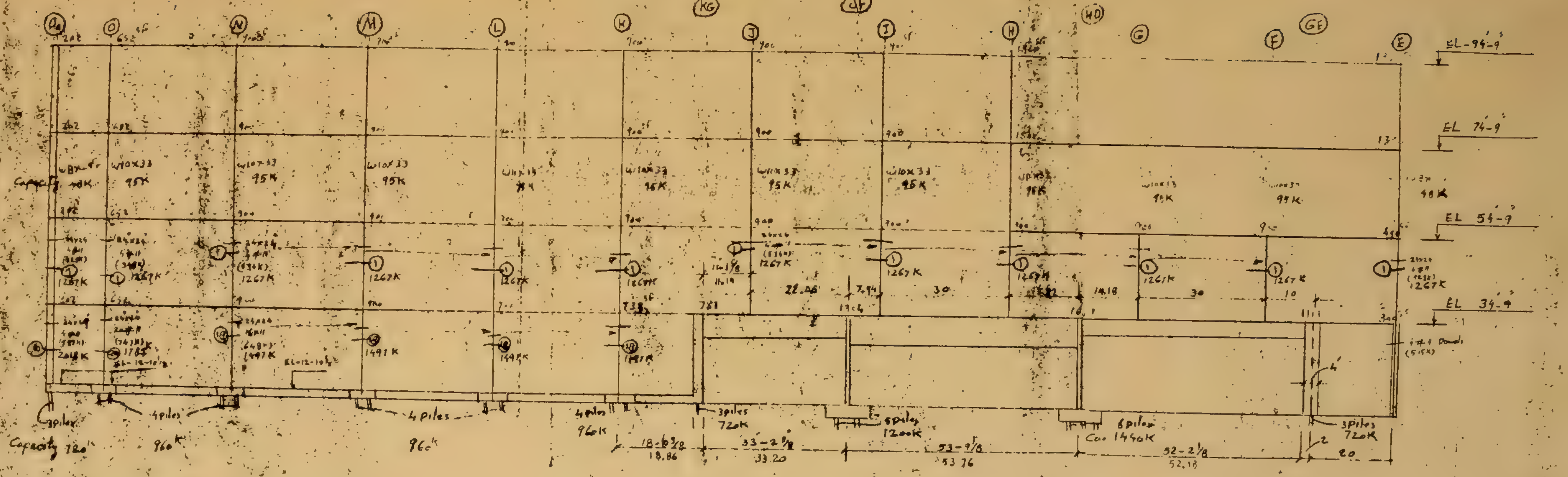


Concrete 5000 PSI @ 28 Days.
Steels to be A-15 Hard grade billet steel 50 ksi
All cols Ties to be #3 @ 12" o.c. unless otherwise.

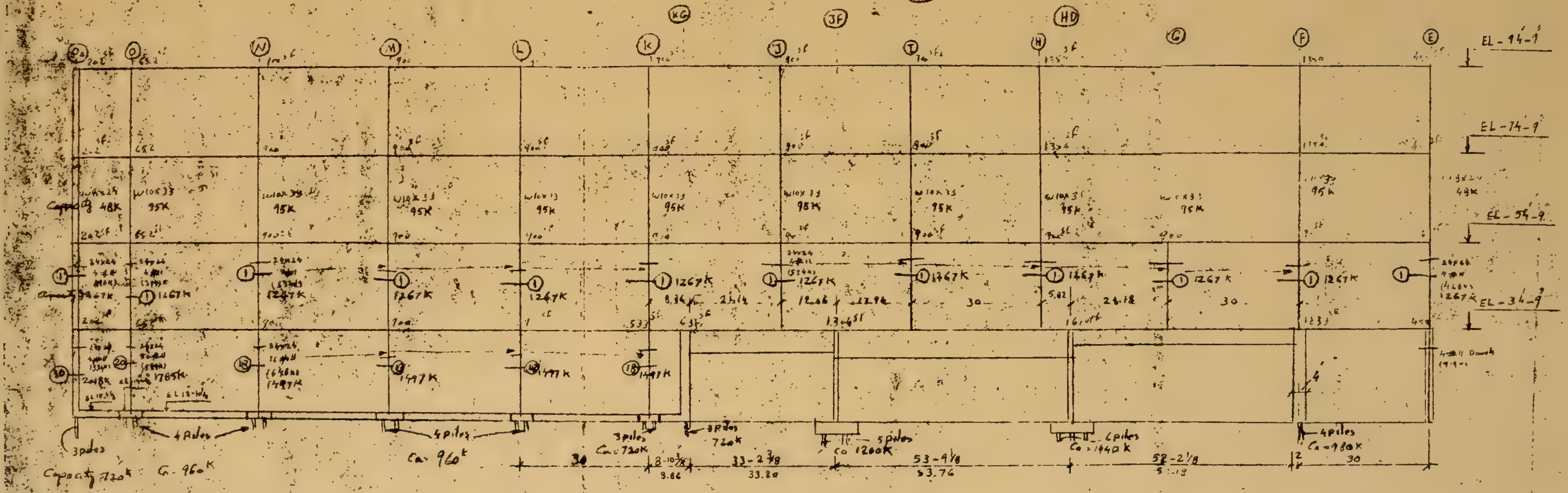
Sect. on line 19



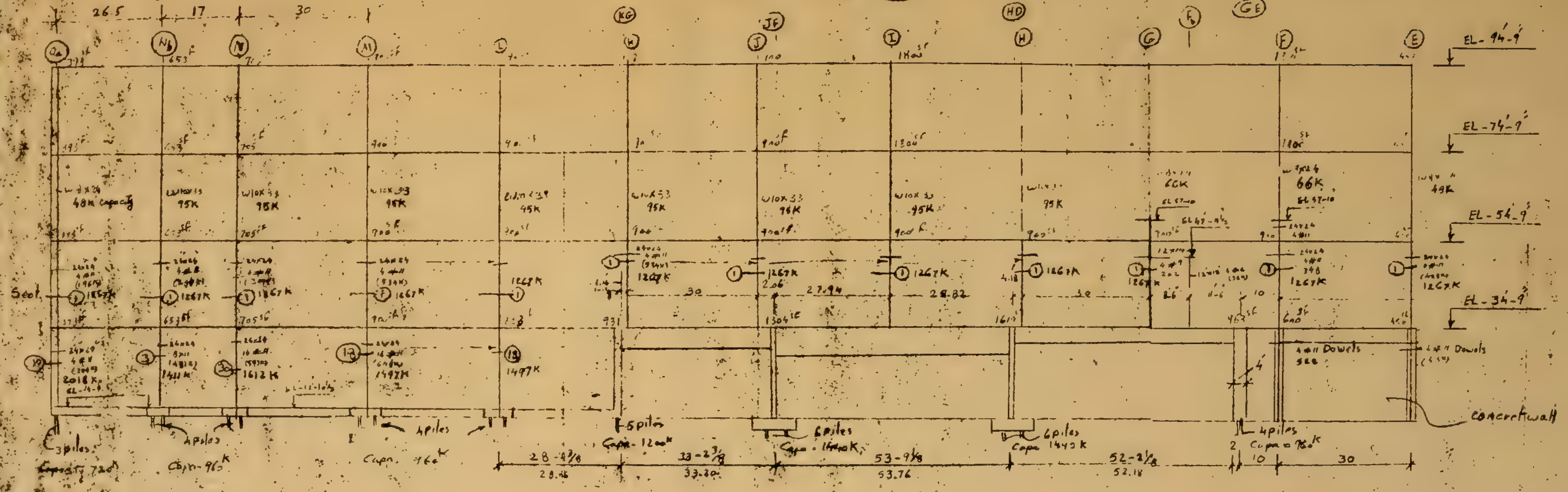
Sect. on line (20)

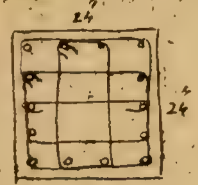


Sect. on line 21



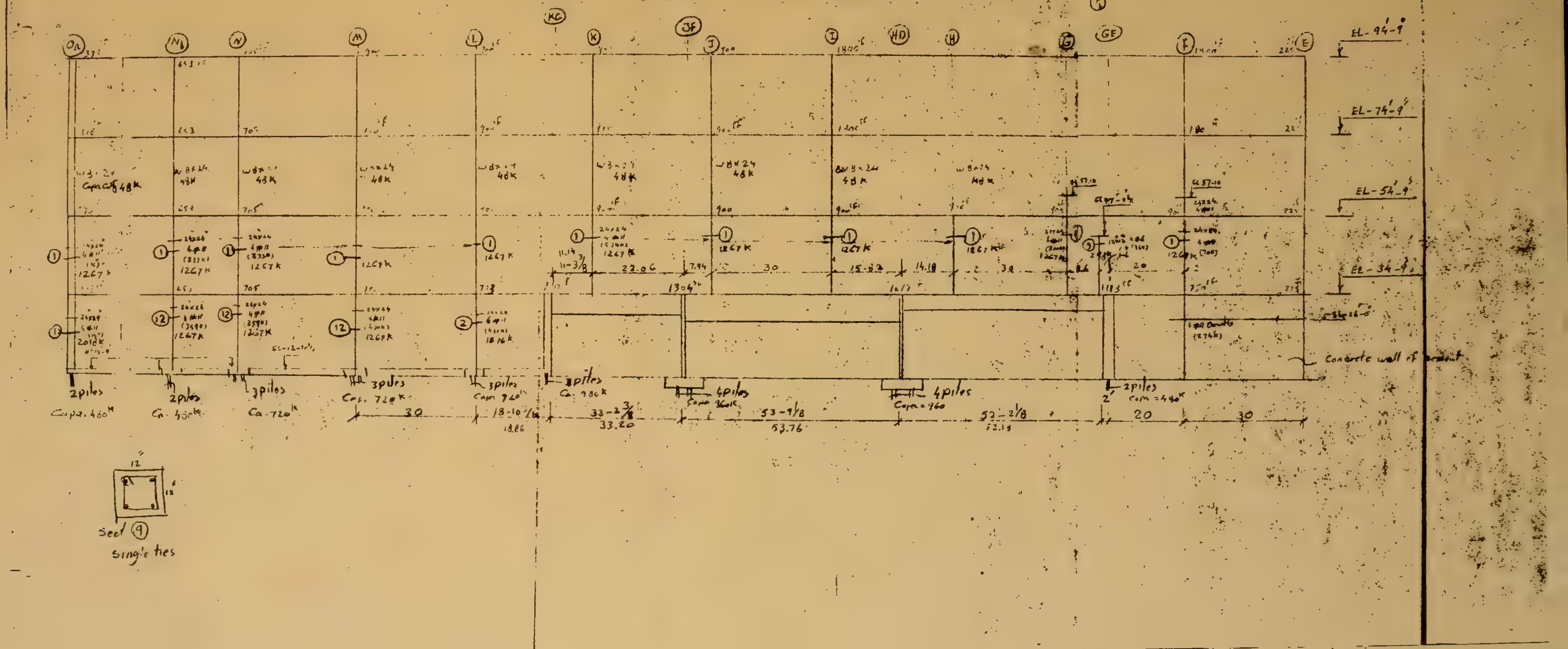
Sect. on line 22





Sect 30
4Ties per set

Sect. on line (22a)



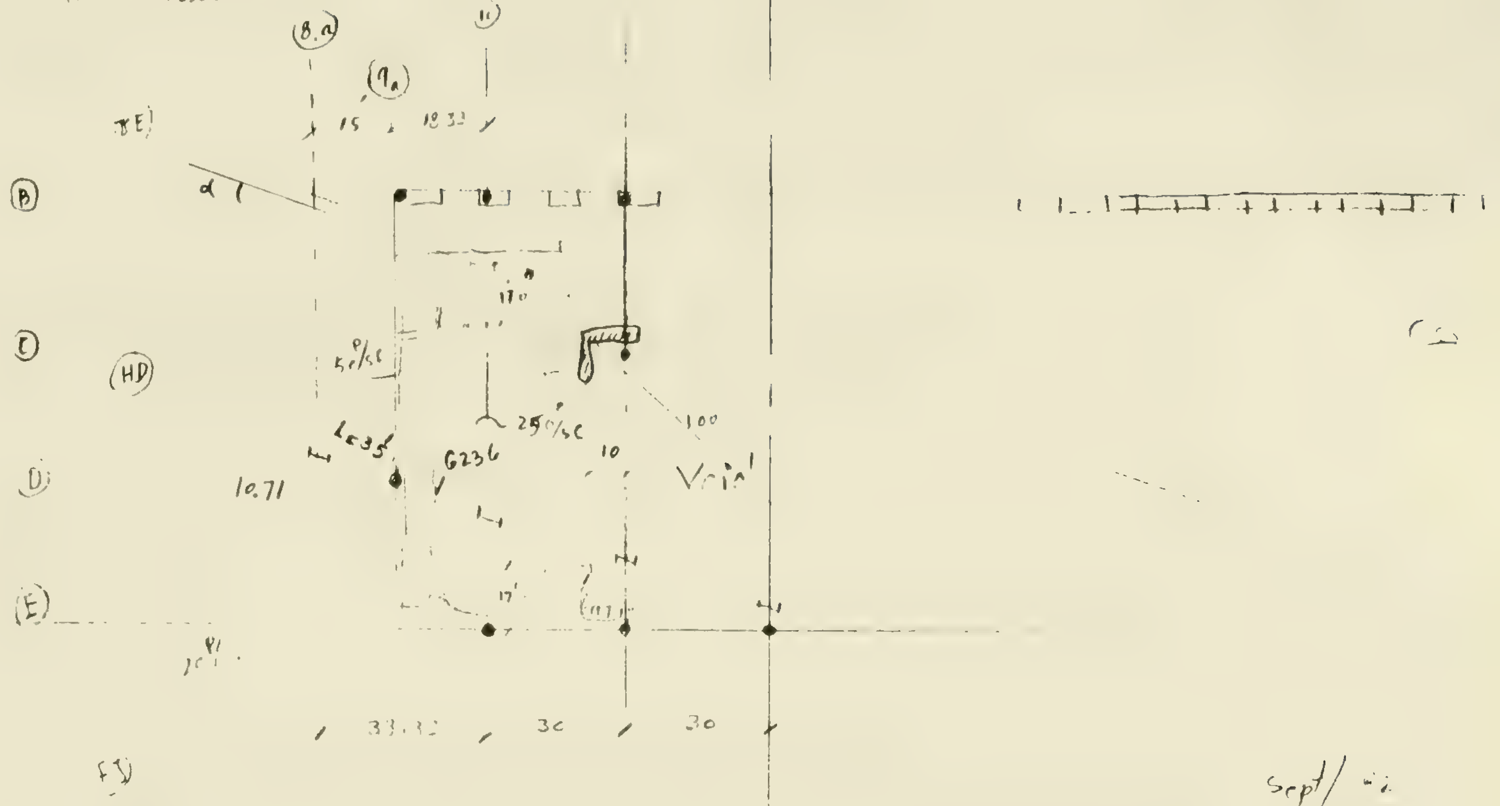
Structural Calculations

Miscellaneous Details and Calculations

Weidlinger Associates

(1)

$\text{Tang}\ \alpha = \dfrac{30}{93.33} = 0.321\ldots \qquad \alpha = 17\ldots$

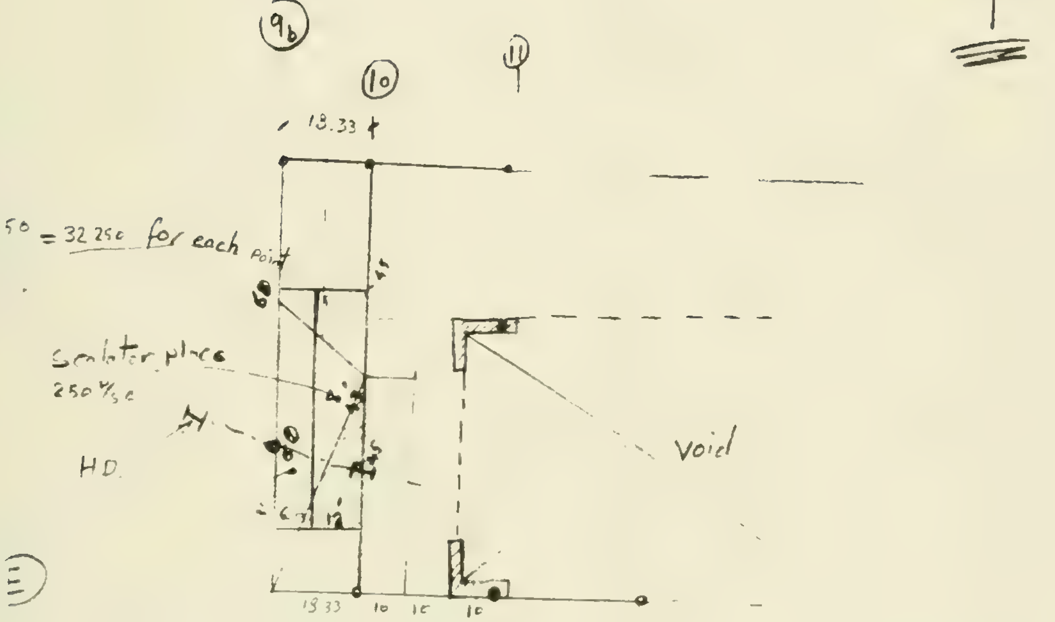


sept/...

146

2

$4 \times 12 \times 250 = 32250$ for each point



$A = \frac{60 \times 18.33}{2} = 550 \times \overset{\text{Roof}}{100} = 55000^{\#} = 55^{k}$ | Load ... / Unit Load

Total load $200\ \text{\#/sf}$ (scalator + others) | $94' - 34' = 60'$ ...

110^{k} for each floor | $60 \times 50 = 3000\ \text{\#}/...$

3 floor $\times 3 = 330^{k}$ | For $60'$ width ...

Roof 55 | $60 \times ...$

Load from slabs 385^{k}

180^{k}

565^{K} Load of Col 9_b and HD

Col. E10 $= A = \frac{10+18.33}{2} \times 45 = 637.65$ sf

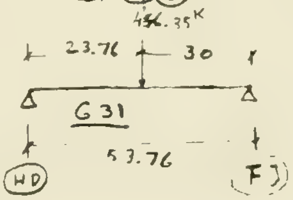
Typ. Floor: $637.65 \times 200 = 127.53 \times 3 = 382.59$

$637.65 \times 100 =$ Roof 63.76

446.35 on G 31

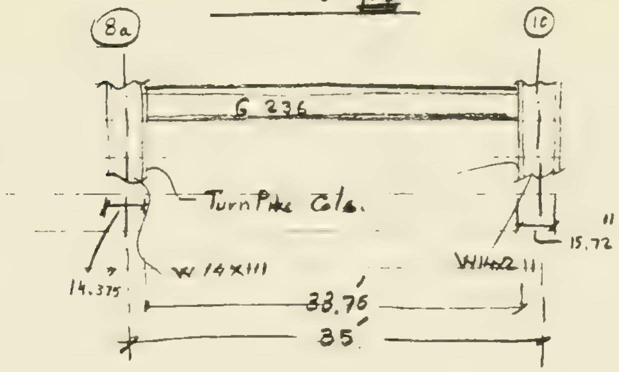
Col. 10 E

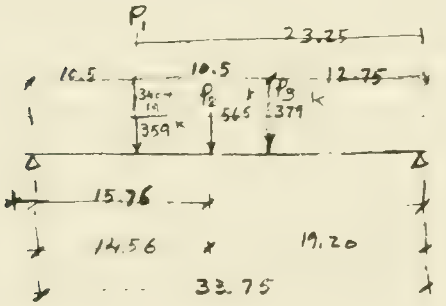
$R_{HD} = \frac{30 \times 446.35}{53.76} = 249.08^{K}$ on HD-10

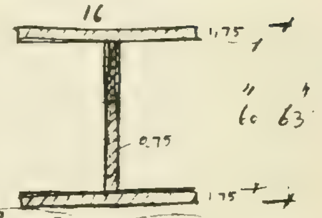


3

Col on line HD







$$I = \underbrace{\frac{16(63^3 - 60^3)}{12}}_{45296} + \underbrace{\frac{0.75 \times 60^3}{12}}_{13500} = 1869^{in}$$

$I = 58896^{in^4}$ $S = 1869$

Col. on HD - 8a

$$R_1 = \begin{cases} R_{P_1} = \dfrac{359 \times 23.25}{33.75} = 247.31 \\ R_{P_2} = \dfrac{565 \times 19.20}{33.75} = 321.42 \\ R_{P_3} = \dfrac{379 \times 12.75}{33.75} = 143.17 \end{cases} \quad 711.90^k$$

$$R_2 = \begin{array}{r} 359 \\ 565 \\ 379 \\ \hline 1303 - \\ 712 \\ \hline 591\,K \end{array}$$

on col. HD - 1c
From Left side

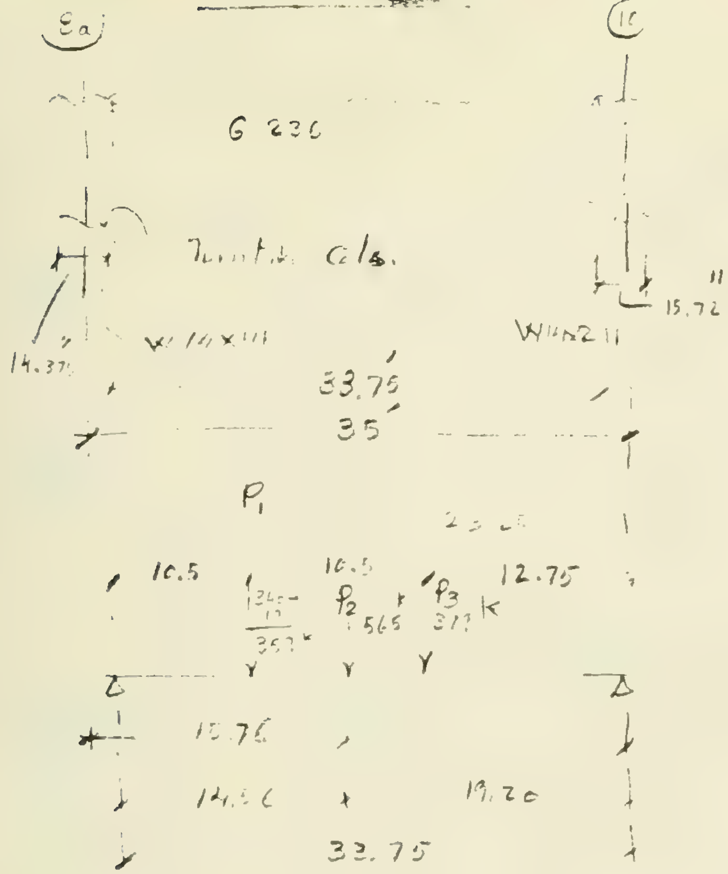
$$M_{\text{Midspan}} = 711.90 \times \frac{33.75}{2} = 12013^{F.k.}$$

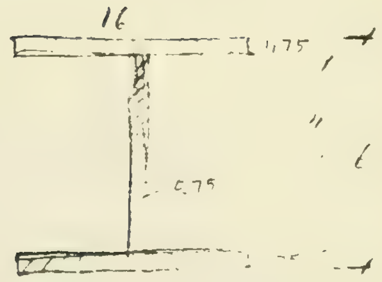
$$\text{required } S_r = \frac{12013 \times 12}{21.5} = 6705^{in^3}$$

$$\begin{array}{r} 247.31 \\ 143.17 \\ \hline 390.48 \end{array}$$

on col. HD - 1c
From Right side

Col on line HD





$I = \frac{16(63^3 - \ldots)}{\ldots} + \frac{0.75 \times \ldots}{12} = \ldots$

$I = \ldots \qquad S = \ldots$

$$R_1 = \begin{cases} R_{P_1} = \dfrac{359 \times 23.25}{33.75} = 247.31 \\ R_{P_2} = \dfrac{565 \times 19.20}{33.75} = 321.42 \\ R_{P_3} = \dfrac{379 \times 12.75}{33.75} = 143.17 \end{cases} \quad 711.90^{k}$$

Col. on HD-8a

$R_2 = $ 359, [illegible], [illegible], ... 712, ... 1 k

on col. HD-... From Left

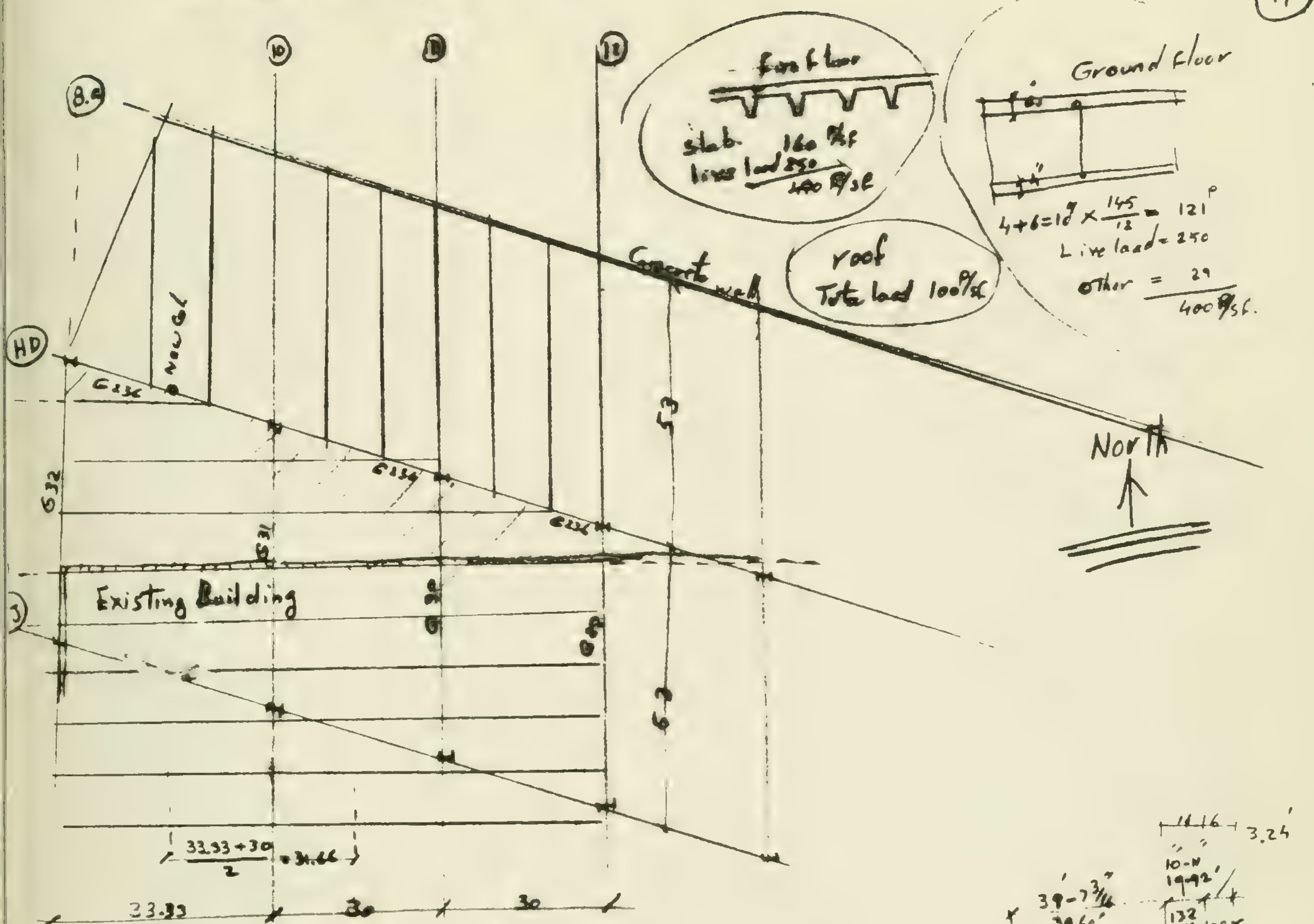
$$M_{\text{Midspan}} = 711.90 \times \frac{33.75}{2} = 12013 \text{ F.k.}$$

$$\text{required } S_r = \frac{12013 \times 12}{21.5} = 6705 \text{ in}^3$$

247
143
390
on col. HD-...
From Right

Loads from Existing bulding

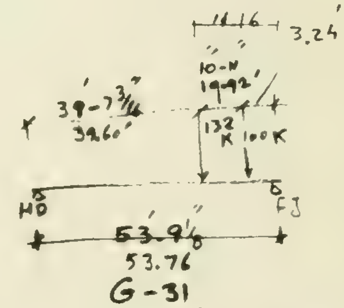
4



Load from G32 to Col (HD-8a)

Ground F. $\frac{33.33}{2} \times \frac{53}{2} \times 4 \times 400$ = 176.65[k] } on col (8.a) & (HD) 217.45 k

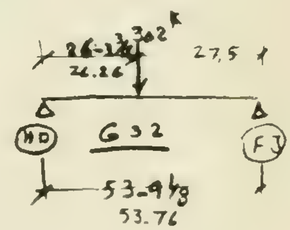
Exist Build. $\frac{132 \times 14.16}{53.76} + \frac{100 \times 3.24}{53.76}$ = 40.80



Load From G31 to Col. (HD-10)

Floor $\frac{31.66 \times 53.76 \times 400^k}{4}$ = 170 k } 324.50[k] on col. (HD)-(10)

Exist. Build. $\frac{302 \times 27.5}{53.76}$ = 154.50 k

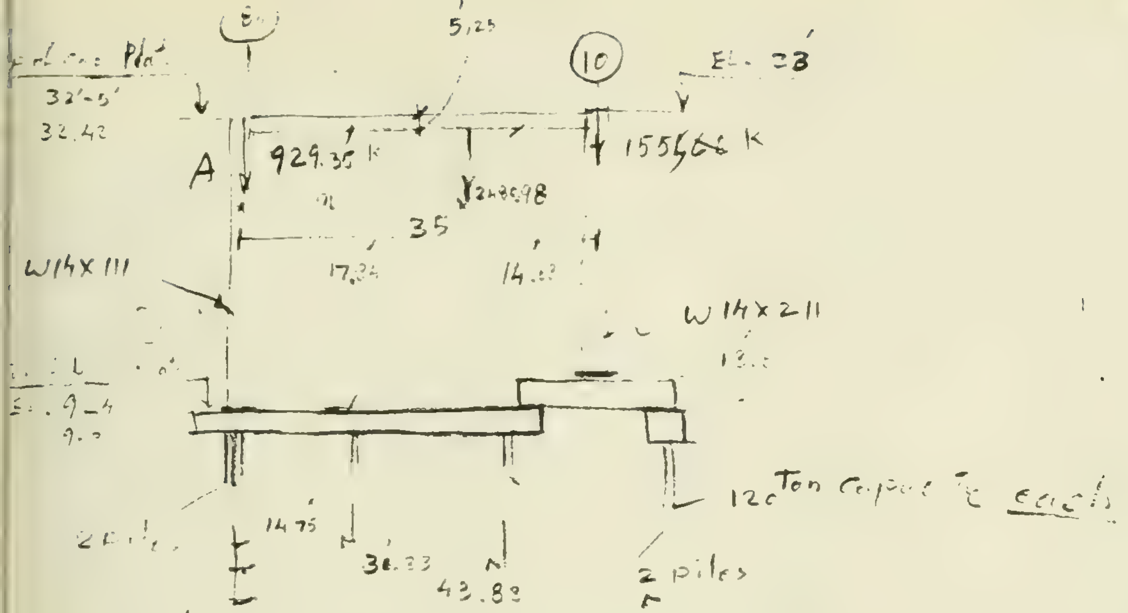


Total Load on Col. (HD-8a) = 711.90 + 217.45 = 929.35 k

Total load on (HD-10) = 591 + 324.50 + 249.08 + 390.48 = 1555.63 k

(151)

5



piles $8 \times 120 \times 2 = 1920$

$1920 \times 8/20 = 384$

2324 < 929.35 + 1551.63 = 2480.98

Gravity Center of Forces

about point A

$929.35 x = 1551.63 \times (35 - x) = 1551.63 \times 35 - 1551.63 x$

f. of l. Loads: $2480.98 x = 1551.63 \times 35$ $\qquad x = 21.89$

For piles: $\overset{21038.4}{2 \times 2 \times 120 \times 43.83} + \overset{14558.4}{2 \times 2 \times 120 \times 30.33} + \overset{7080}{2 \times 2 \times 120 \times 14.75} = 2324 \bar{X}$

$$\bar{X} = \frac{\overset{42676.8}{21038.4 + 14558.4 + 7080}}{2324} = 18.36$$

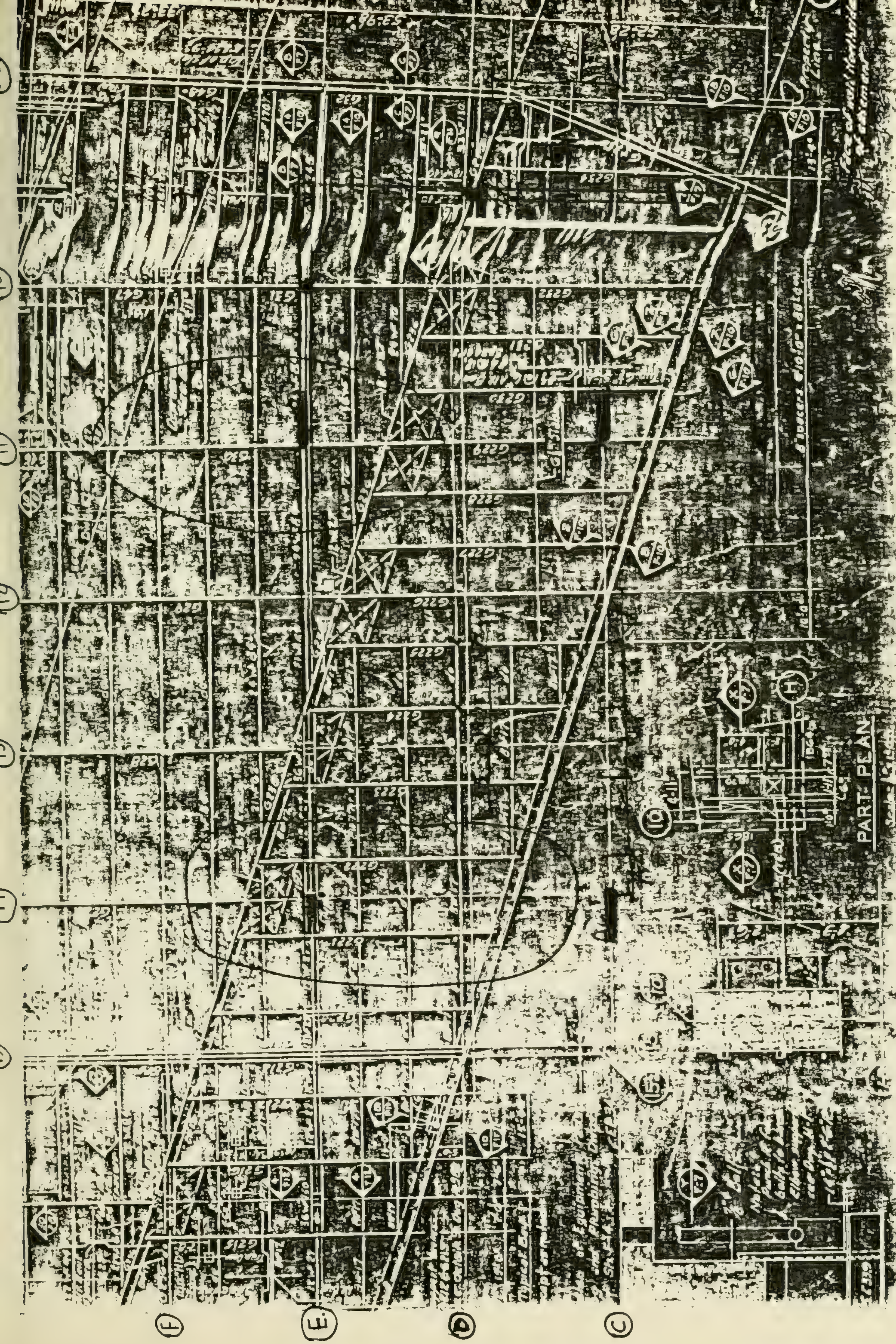
21.89

2480.98

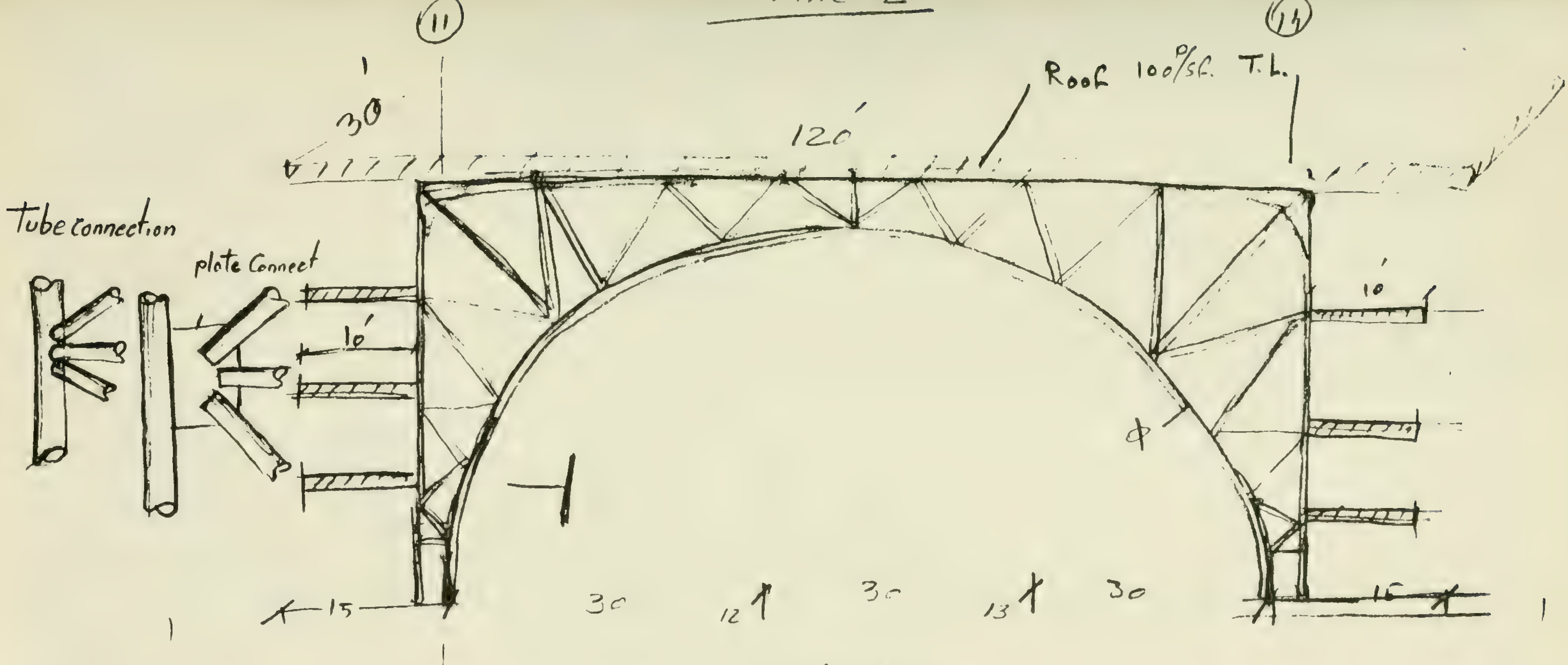
2324

18.36

3.53


PART PLAN
F
E
D
C

on Line E



Roof D.L & L.L.

on line E

$120 \times 30 = 3600^{sf} \times 100^{\#/sf} = 360{,}000 \div 2 = 180$ for each support

Aprox. { Exist. First Floor $\times 30 \times 15 = 450^{sf} \times 400^{\#/sf} = 180{,}000$ = 180

Exist Roof $30 \times 15 = 450 \times 100^{\#/sf} = 45{,}000$ = 45

405^{K}

on line C

Roof { $60 \times 30 = 2100 \times 100 = 210^{K}$

$60 \times 10 = 600 \times 300 = 180{,}000^{K}$

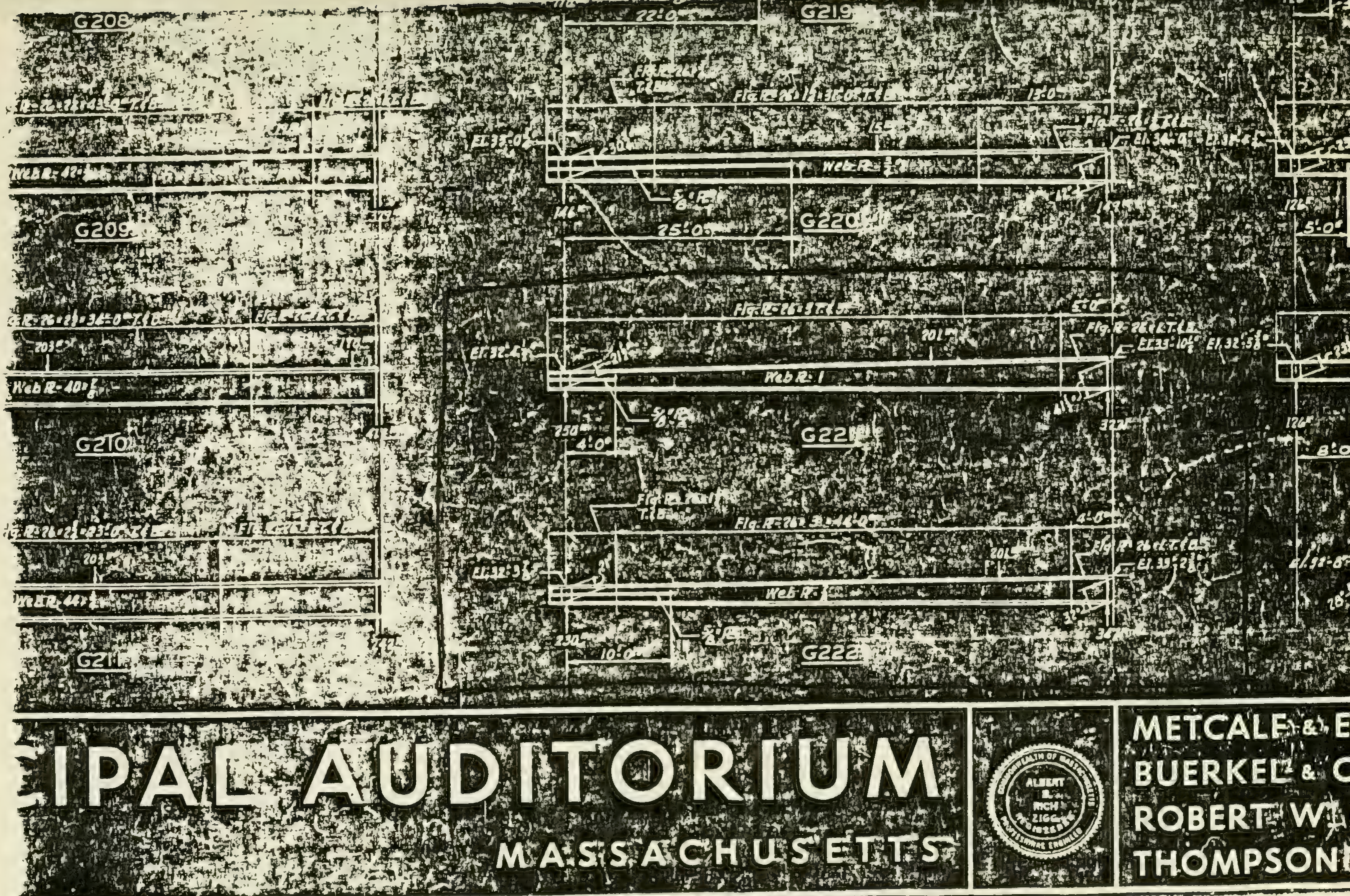
side slabes $10 \times 30 + 10 \times 3 \times 200 =$

240,00

side slabes for 3 level.
$3 \times 10 \times 30 \times 200 = 180{,}000^{K}$

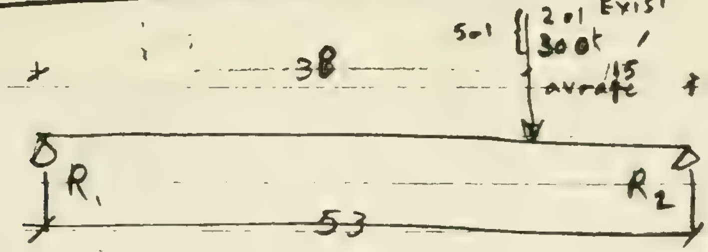
405 +
180
$585^{K} \rightarrow 600^{K}$

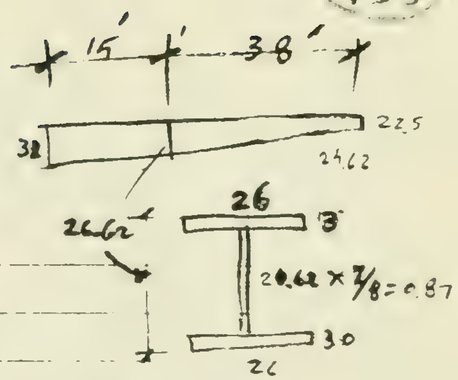
153


G208
G209
G210
G211
G219
G220
G221
G222
22'-0"
25'-0"
4'-0"
10'-0"
5'-0"
8'-0"
CIPAL AUDITORIUM
MASSACHUSETTS
ALBERT S. RICH ZIGG
METCALF & E
BUERKEL & C
ROBERT W
THOMPSON

154

Check G 221 & G 222



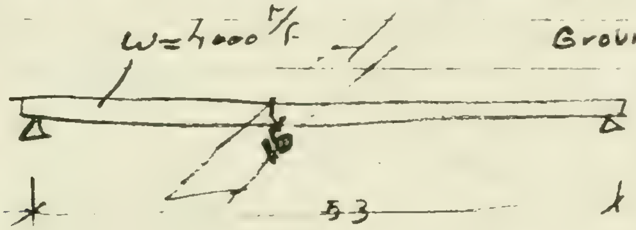


21875 639

$$I = \frac{26\,(26.62^3 - 20.62^3)}{12} + \frac{0.875 \times 20.62^3}{12} = 22514\ \text{in}^4 \qquad S = 1691\ \text{in}^3$$

$$R_1 = \frac{501 \times 15}{53} = 141.80^k \qquad 501 - 141.80 = 368.20^k$$

$$M = 141.80 \times 38 = 5388.40 \div 2 = 2694^{F.K} \text{ for each Gi}$$



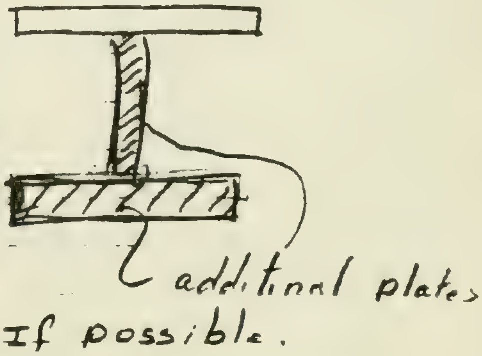
Ground floor { slab 150, Live load 250 — 400

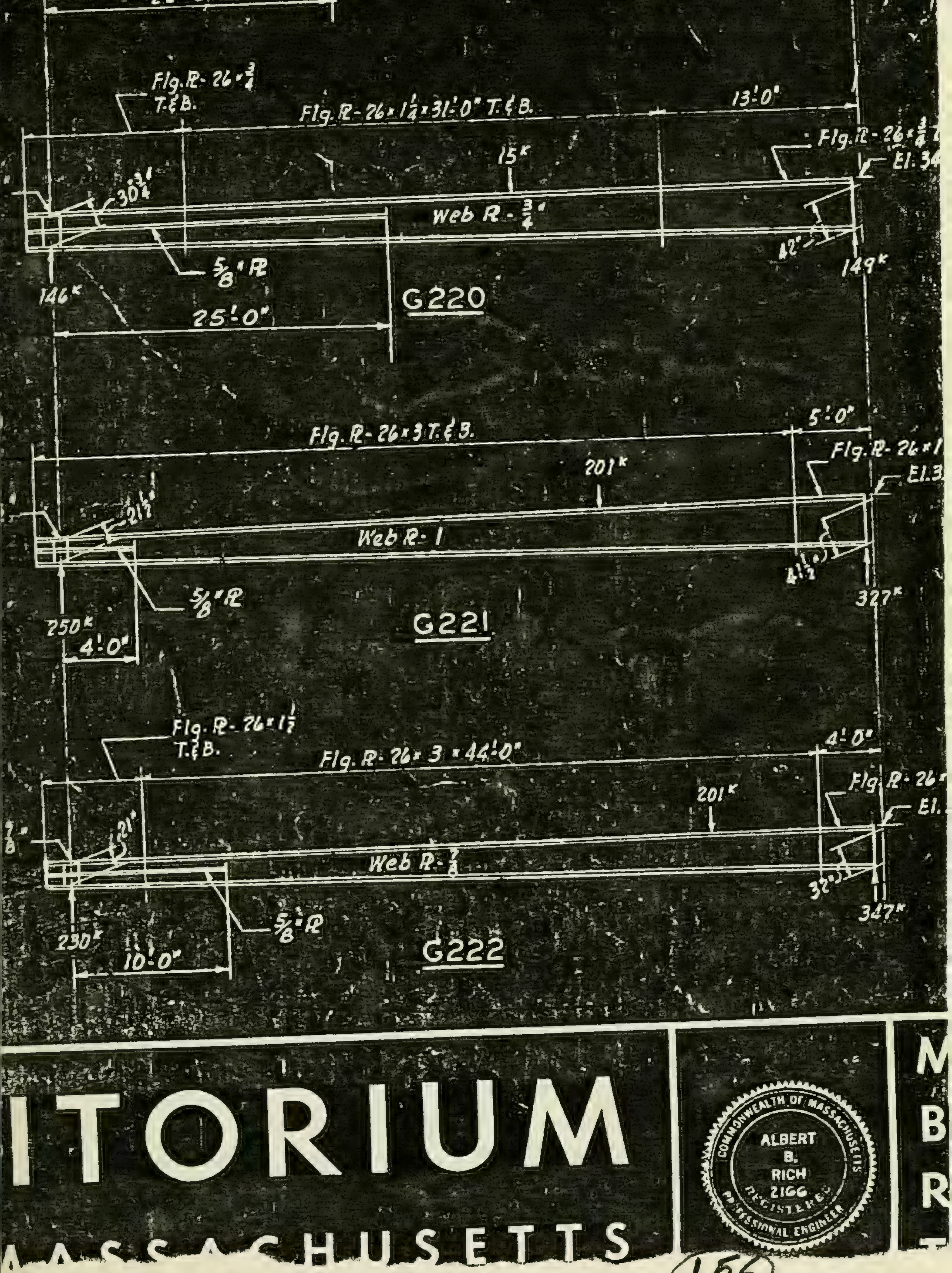
$$M_2 = \frac{6 \times 53^2}{8} = \boxed{2100} \qquad \frac{1405 \times 12}{21.5} = 700$$

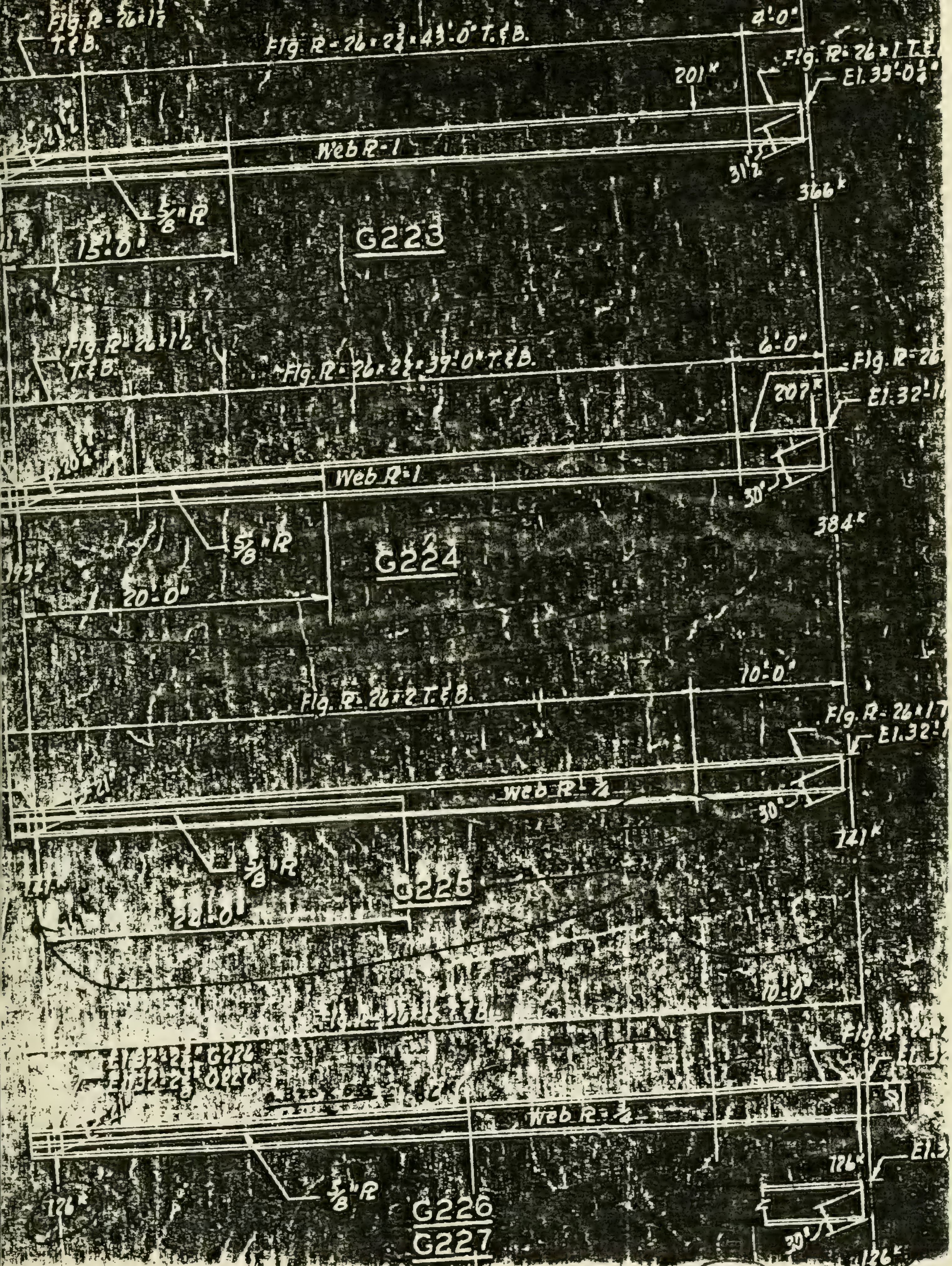
$$2100 + 2694 = \boxed{4794}$$

$$\frac{4794 \times 12}{21.5} = 2676\ \text{in}^3 - 1691 = 985 \pm\ \text{in}^3$$

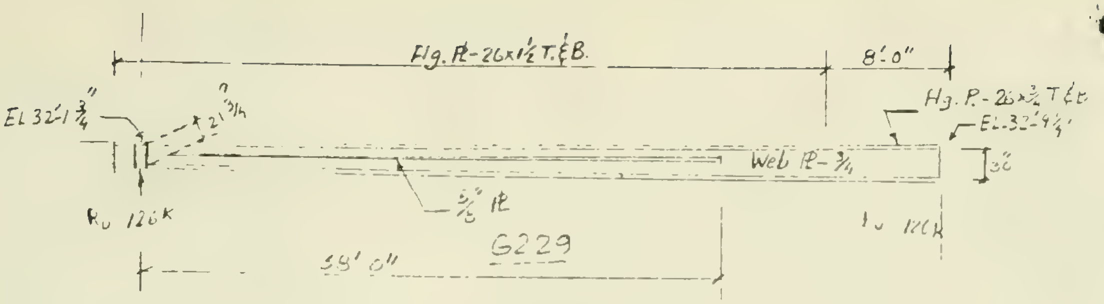
To be reinforced Girders. 221 & 222

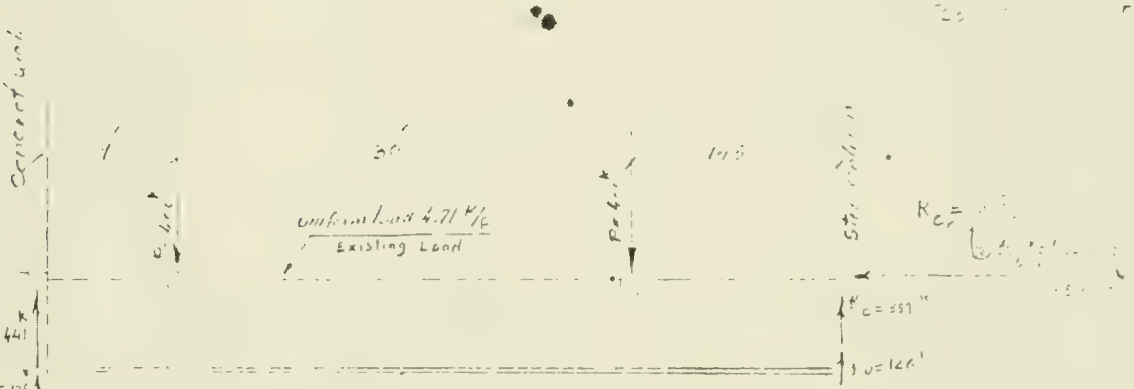



Flg. R - 26 × 3/4 T.&B.
Flg. R - 26 × 1 1/4 × 31'-0" T.&B.
13'-0"
Flg. R - 26 × 3/4
El. 34
15k
30 3/4"
Web R - 3/4"
5/8" R
42"
149k
146k
G220
25'-0"
Flg. R - 26 × 3 T.&B.
5'-0"
Flg. R - 26 × 1
El. 3
201k
21 1/2"
Web R - 1
4 1/2"
5/8" R
327k
250k
4'-0"
G221
Flg. R - 26 × 1 1/2 T.&B.
Flg. R - 26 × 3 × 44'-0"
4'-0"
Flg. R - 26
El.
201k
21"
7/8"
Web R - 7/8
32"
347k
5/8" R
230k
10'-0"
G222
ITORIUM
MASSACHUSETTS
COMMONWEALTH OF MASSACHUSETTS
ALBERT B. RICH
2166
REGISTERED
PROFESSIONAL ENGINEER
M
B
R



Flg. R - 26 x 1½ T.&B.
Flg. R - 26 x 2¼ x 43'-0" T.&B.
4'-0"
Flg. R - 26 x 1 T.&B.
201k
El. 33'-0¼"
Web R - 1
31½
366k
5/8" R
15'-0"
G223
Flg. R - 26 x 1½ T.&B.
Flg. R - 26 x 2¼ x 39'-0" T.&B.
6'-0"
207k
El. 32'-11
Web R - 1
30°
384k
5/8" R
G224
20'-0"
10'-0"
Flg. R - 26 x 2 T.&B.
Flg. R - 26 x 1 T
El. 32'-
Web R - ¼
30°
121k
5/8" R
G225
22'-0"
10'-0"
Web R - ¼
126k
5/8" R
G226
G227
30°
126k
86k





$R_{c_1} = 800 - 359 = 441^k$

G229 New proposal plate Girder

$M_p = \frac{441 \times 53.5}{2} - \ldots$

$M_p = 4696.75$ FK

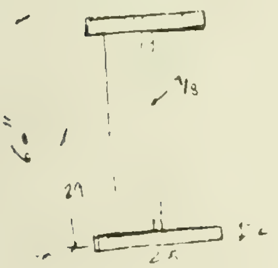
$M_u = \frac{4.71 \times 53.5^2}{8} = 1658.14$

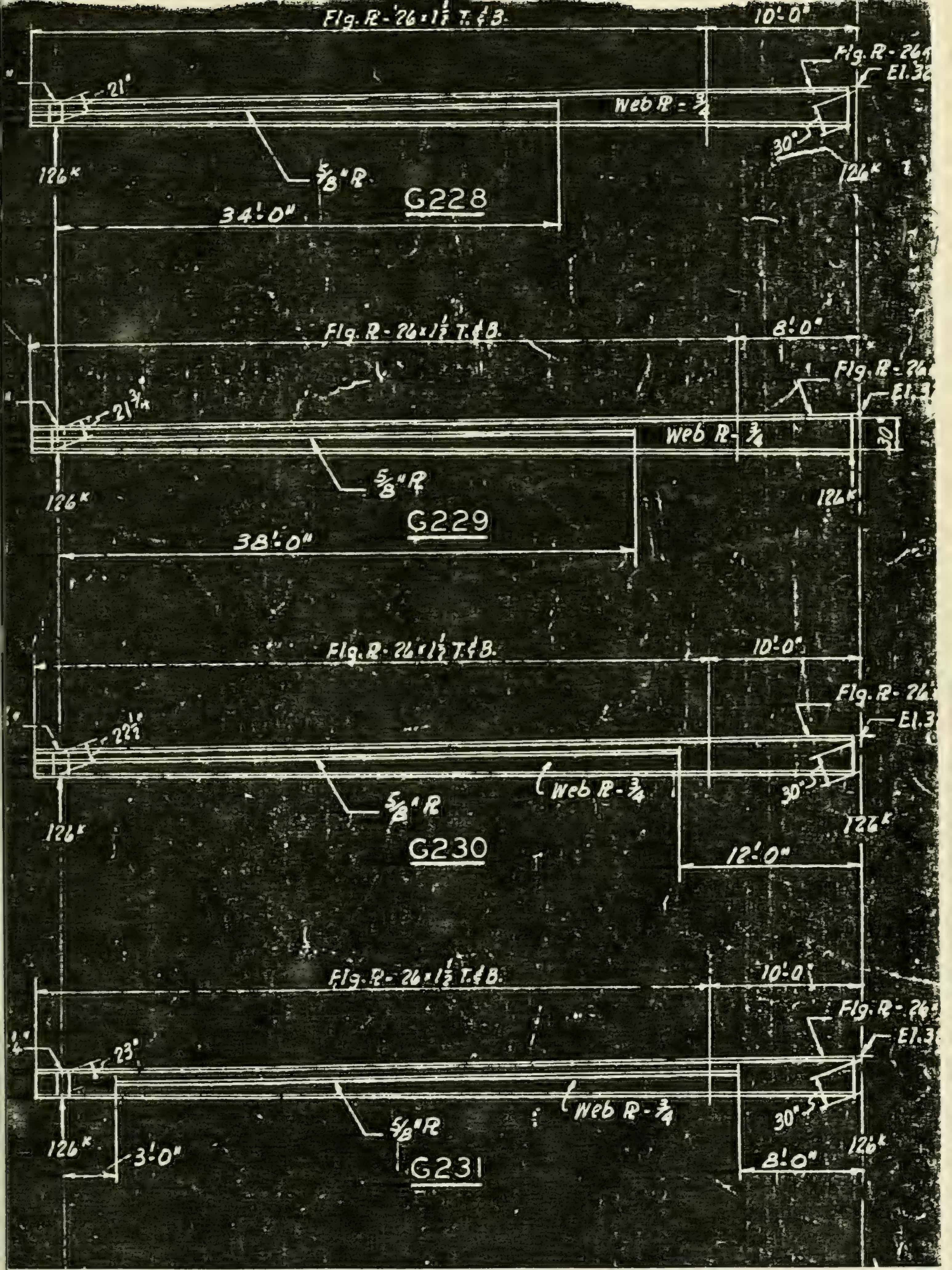
$\Sigma M = 4696.75 + 1658.14 =$

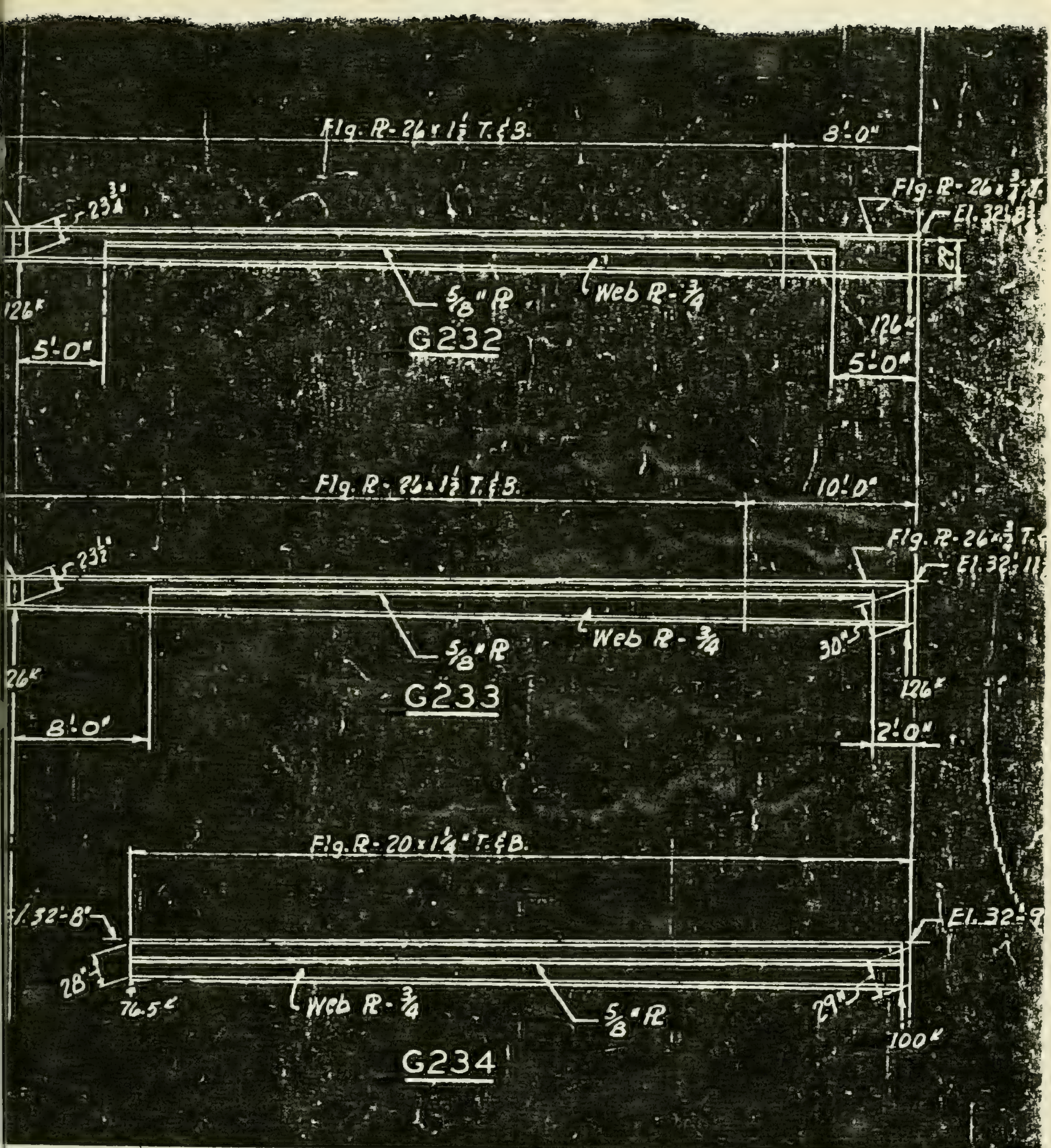
$M = 6382$ F.K.

$S_x = \frac{6382 \times 12}{24} = 3191 \text{ in}^3$

$S = \frac{25(60^3 - 56^3)}{6 \times 60} + \frac{\ldots \times 56^2}{6} = 3261.77$




Flg. R - 26 x 1½ T. & B.
10'-0"
Web R - ¾
5/8" R
126^K
34'-0"
G228
8'-0"
38'-0"
G229
10'-0"
12'-0"
G230
10'-0"
3'-0"
8'-0"
G231
30"

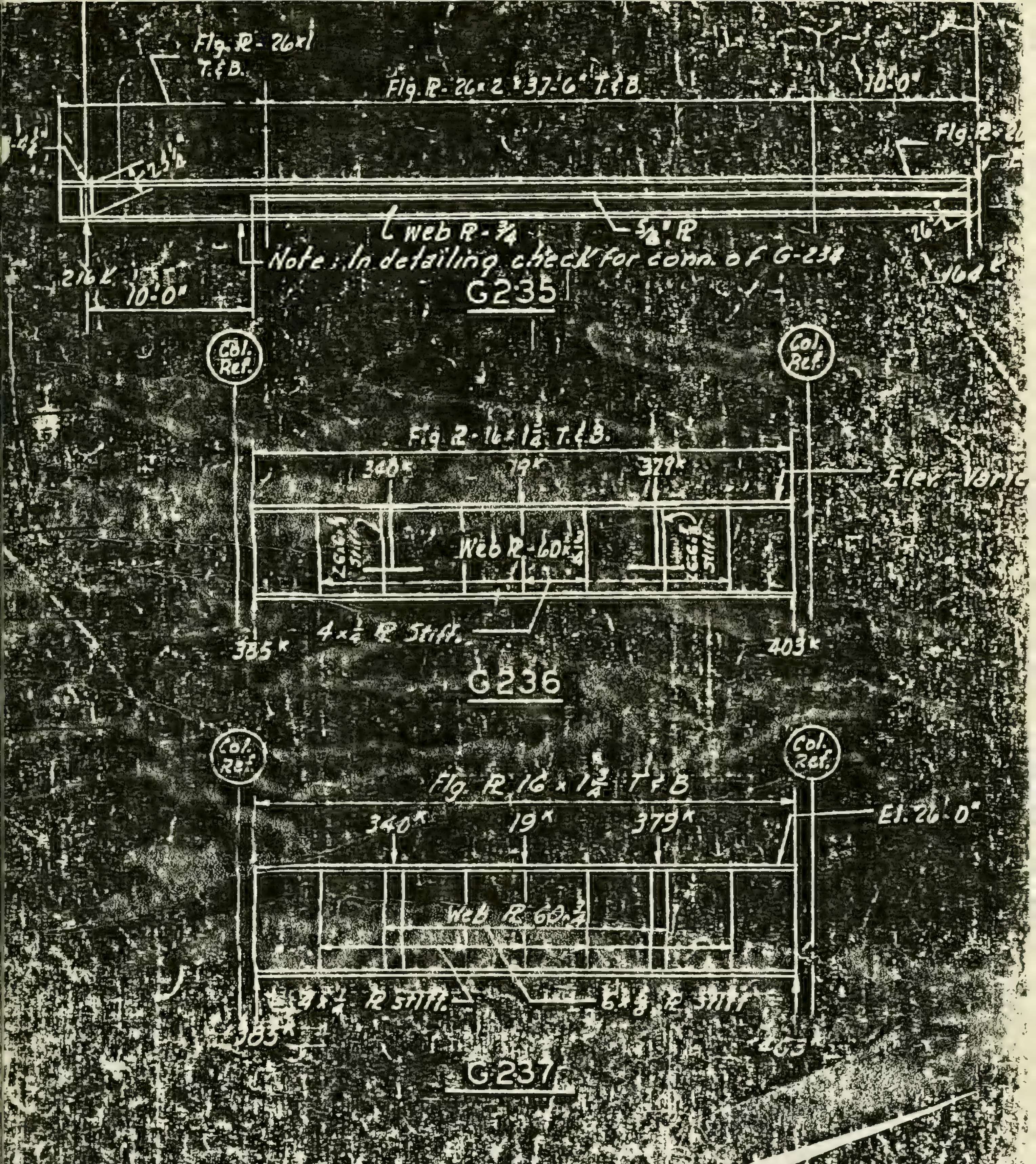


& EDDY, INC. — CIVIL ENGINEERS

& CO., INC. — MECHANICAL ENGINEERS

W. SULLIVAN, INC. — PLUMBING ENGINEERS

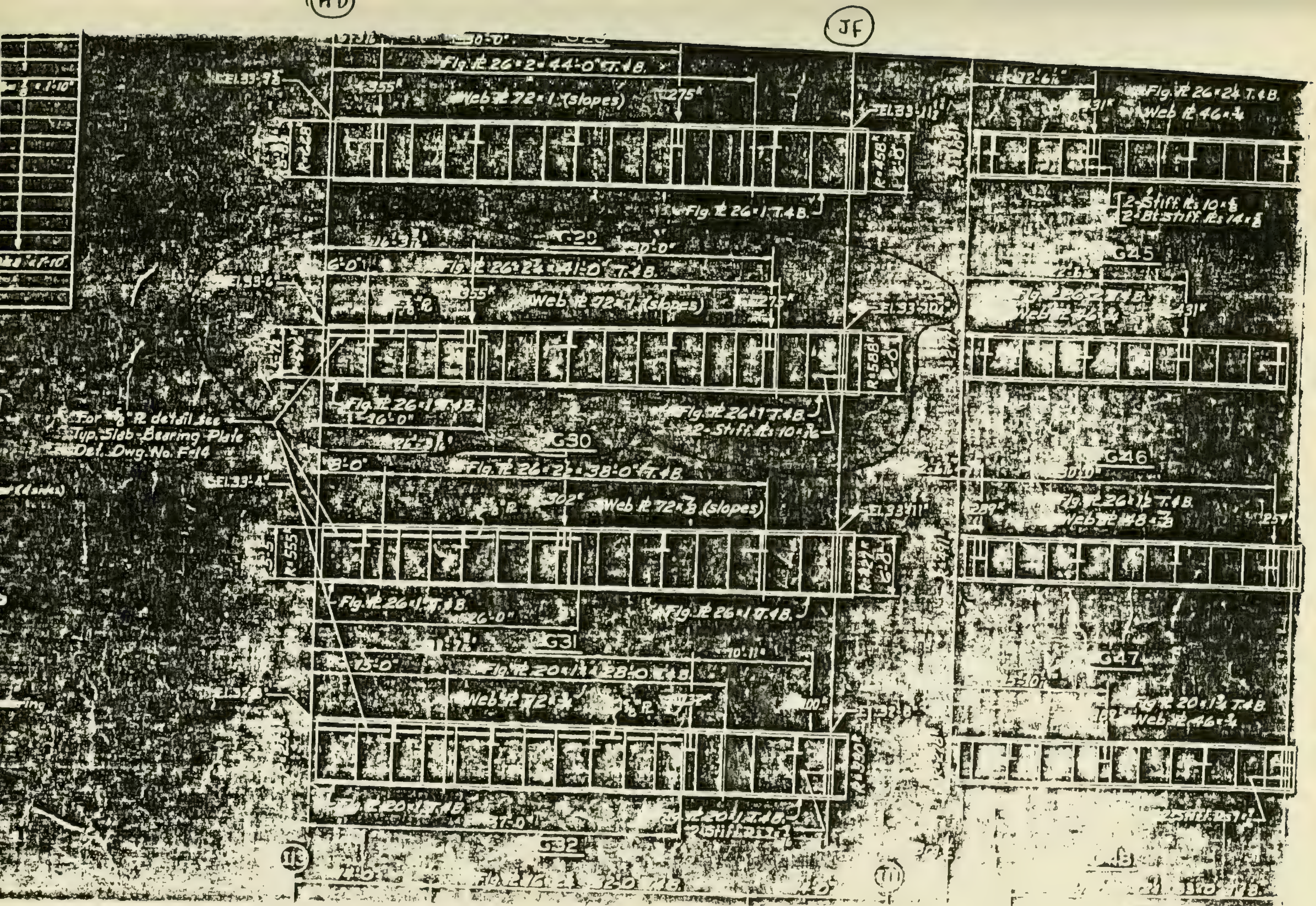
ON ENGINEERING CO. — ELECTRICAL ENGINEERS



Note: Gdrs. 215 to 235 incl. have uniformly sloping Top Flg. R's & flush Bot. R's. Web R depth dims. are taken ℄ Support.

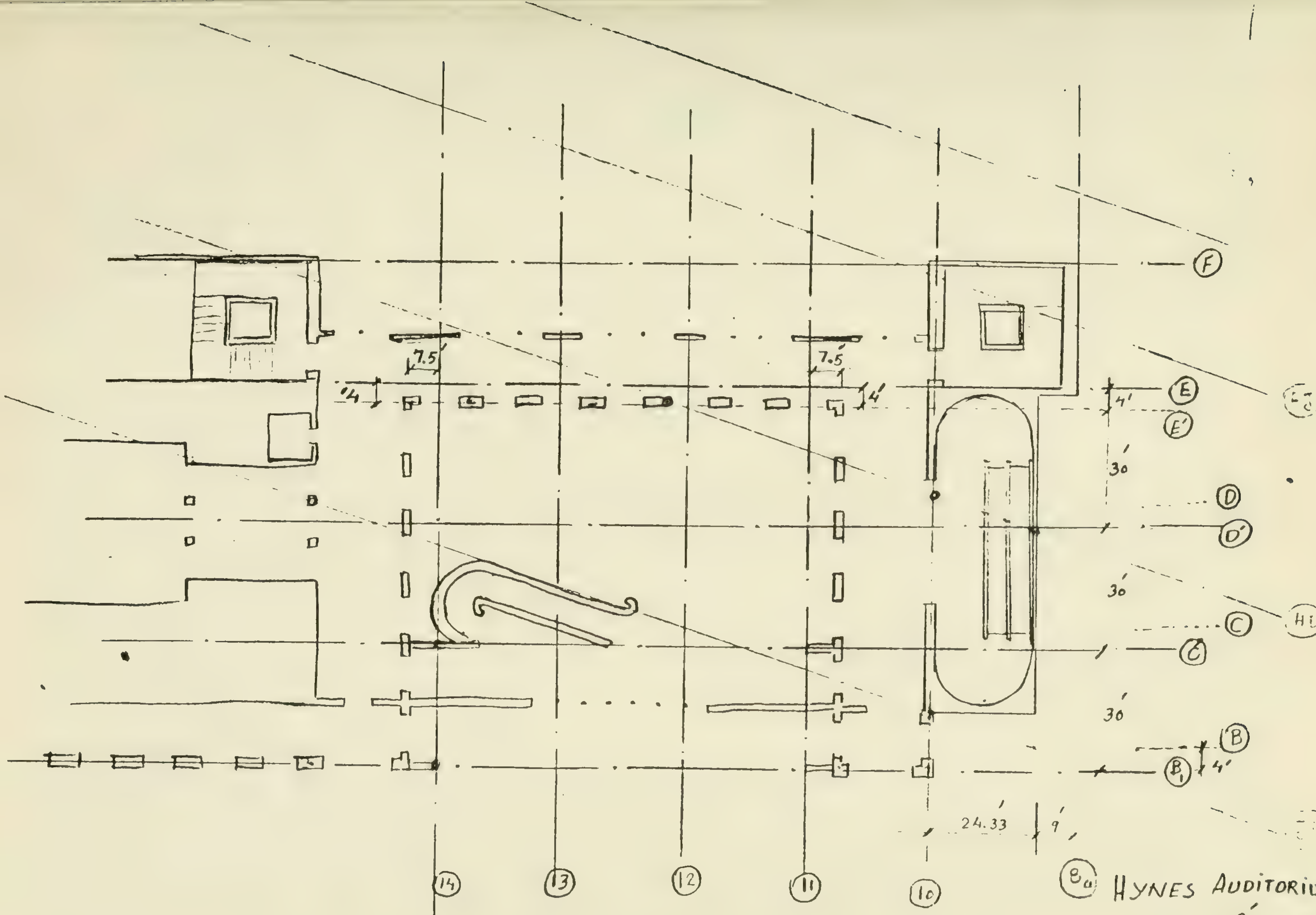
End Stiffeners ... 5/8 fitted, 8" I.F.-T.F. (Typ.- Strut

Intermed. ... ers 6×5/8 fitted, max. spacing 5'-0" (T



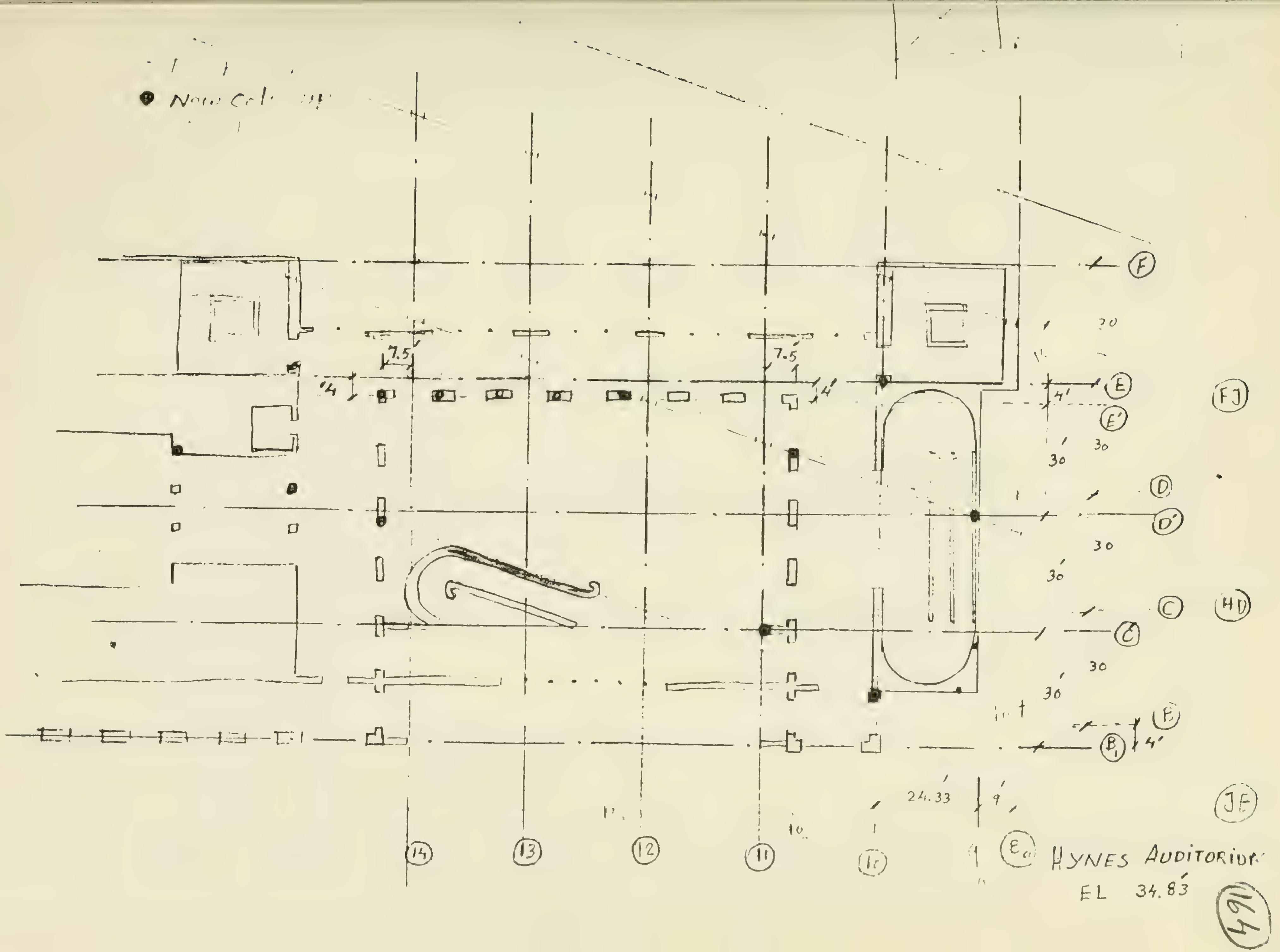
JF
G20
Flg. R 26 x 2 x 44'-0" T.&B.
Web R 72 x 1 (slopes)
355k
275k
Flg. R 26 x 1 T.&B.
G29
Flg. R 26 x 2½ x 41'-0" T.&B.
Web R 72 x 1 (slopes)
Flg. R 26 x 1 T.&B.
16'-0"
For R detail see Typ. Slab-Bearing Plate Det. Dwg. No. F-14
G30
Flg. R 26 x 2 x 38'-0" T.&B.
Web R 72 x ⅞ (slopes)
Flg. R 26 x 1 T.&B.
2-Stiff. Rs 10 x ⅞
26'-0"
G31
Flg. R 20 x 1¾ x 28'-0" T.&B.
Web R 72 x ¾
G32
G45
Flg. R 26 x 2¼ T.&B.
Web R 46 x ¾
2-Stiff. Rs 10 x ⅞
2-Bt. Stiff. Rs 14 x ⅞
G46
Flg. R 26 x 1½ T.&B.
Web R 48 x ⅞
G47
Flg. R 20 x 1¾ T.&B.
Web R 46 x ¾

162

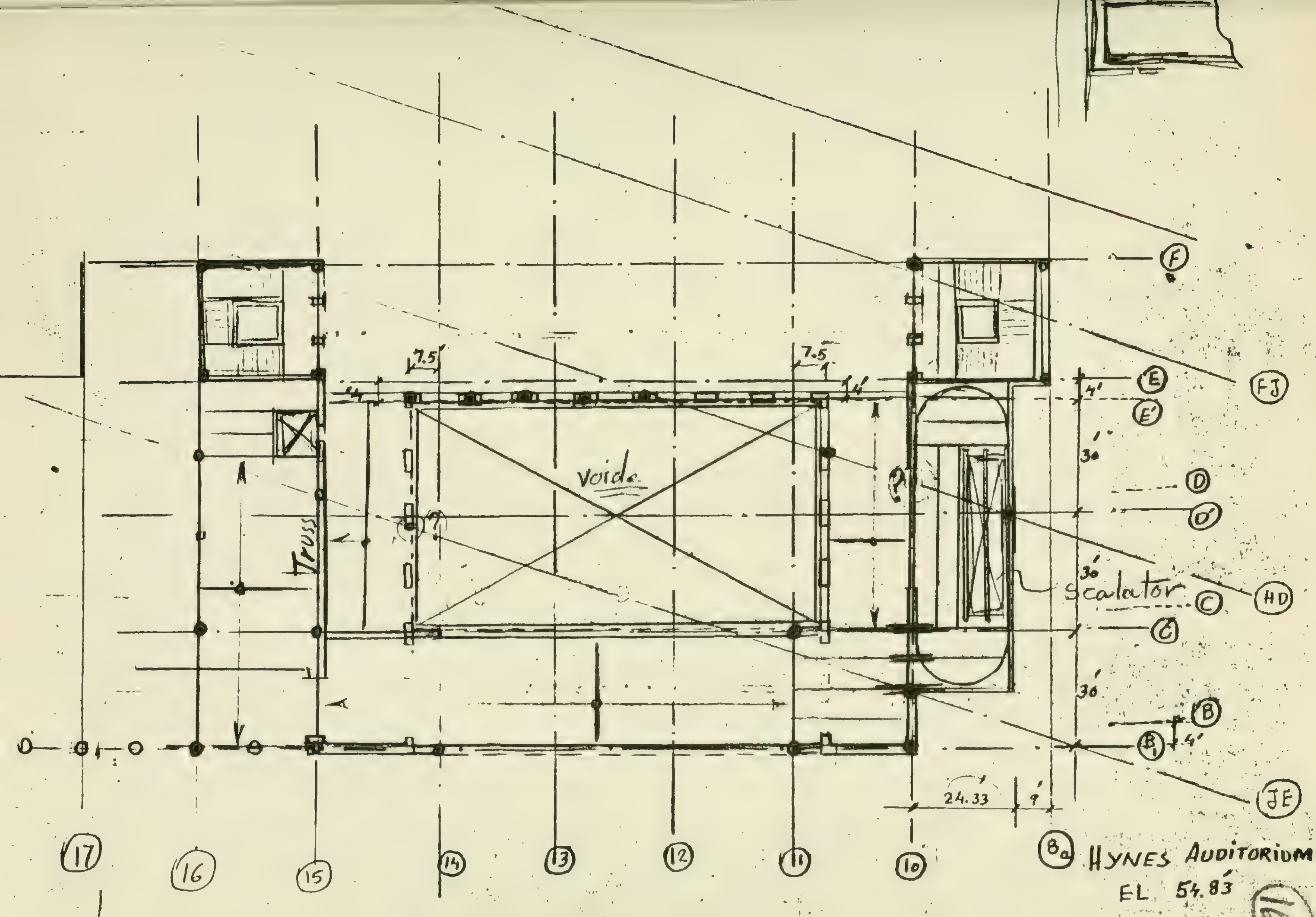


8a HYNES AUDITORIUM
EL 34.83'

163



New Col
7.5
7.5
4
4'
4'
30
30
30
30
30
30
30
24.33
9'
4'
F
E
E'
D
D'
C
C'
E
B1
FJ
HD
JF
14
13
12
11
10
Ea
HYNES AUDITORIUM
EL 34.83'
164

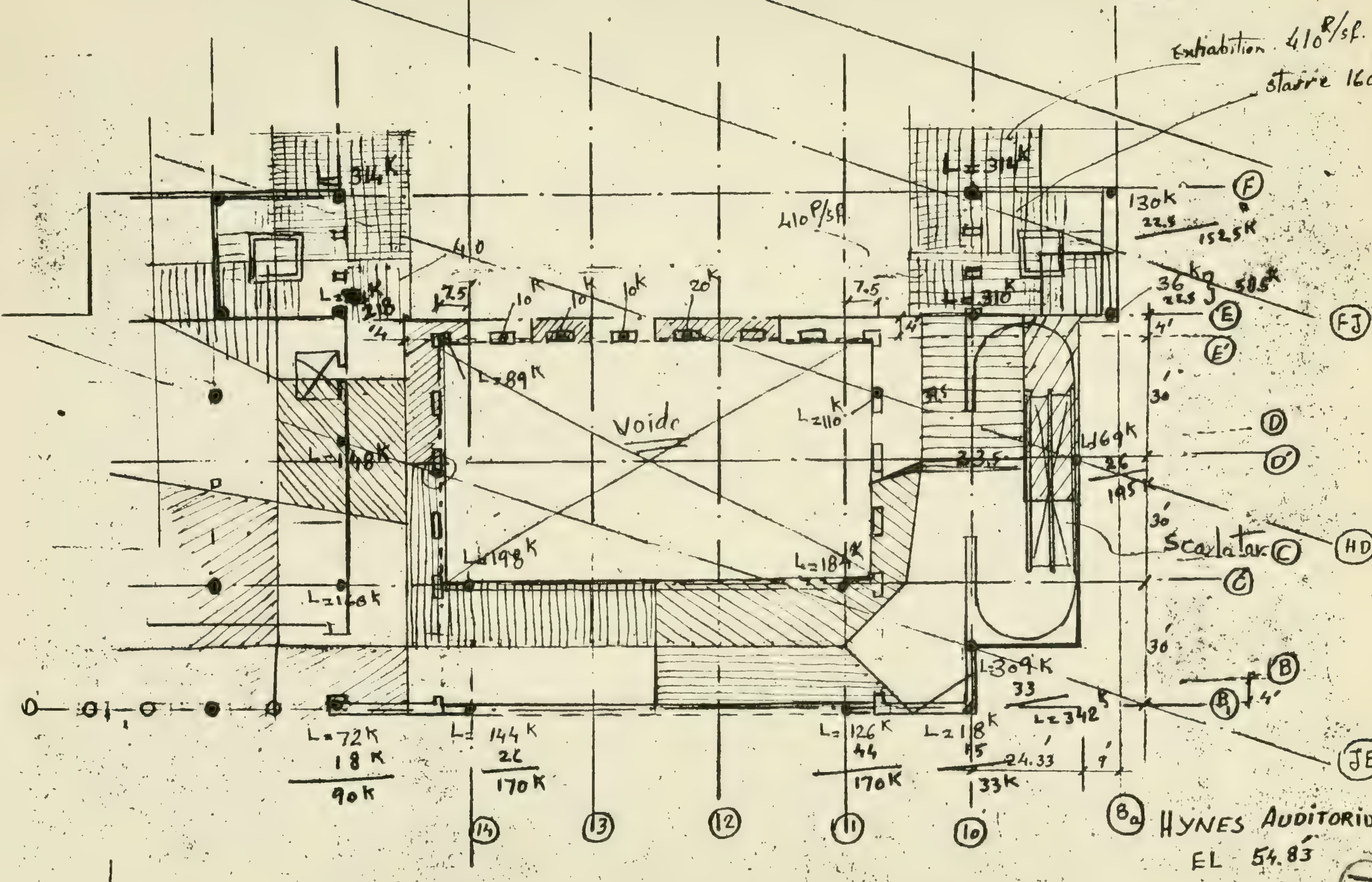


HYNES AUDITORIUM
EL 54.83'

165

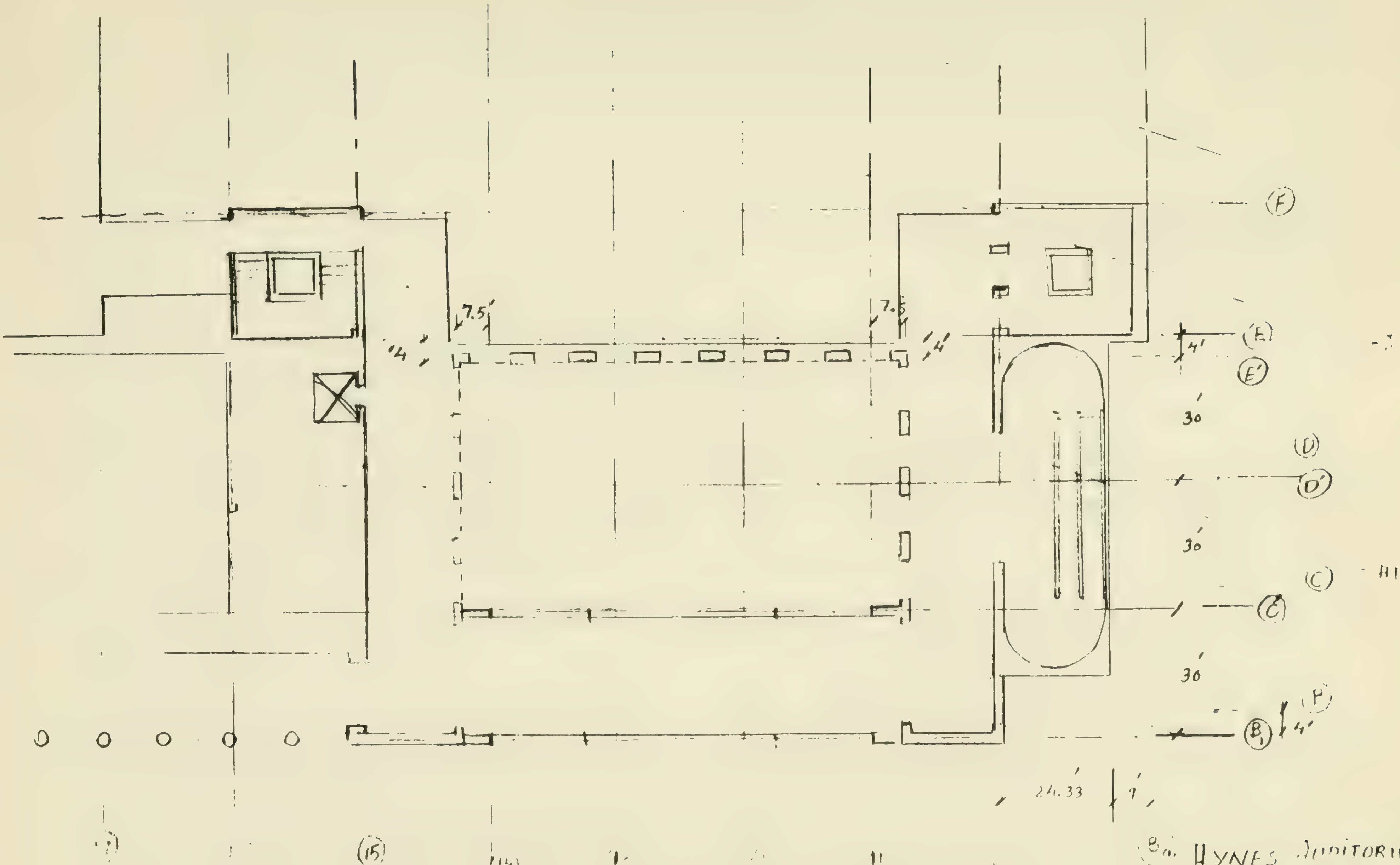
Unit Loads. Exist. Foor T.L 410 P/s

walls.



HYNES AUDITORIUM
EL 54.83

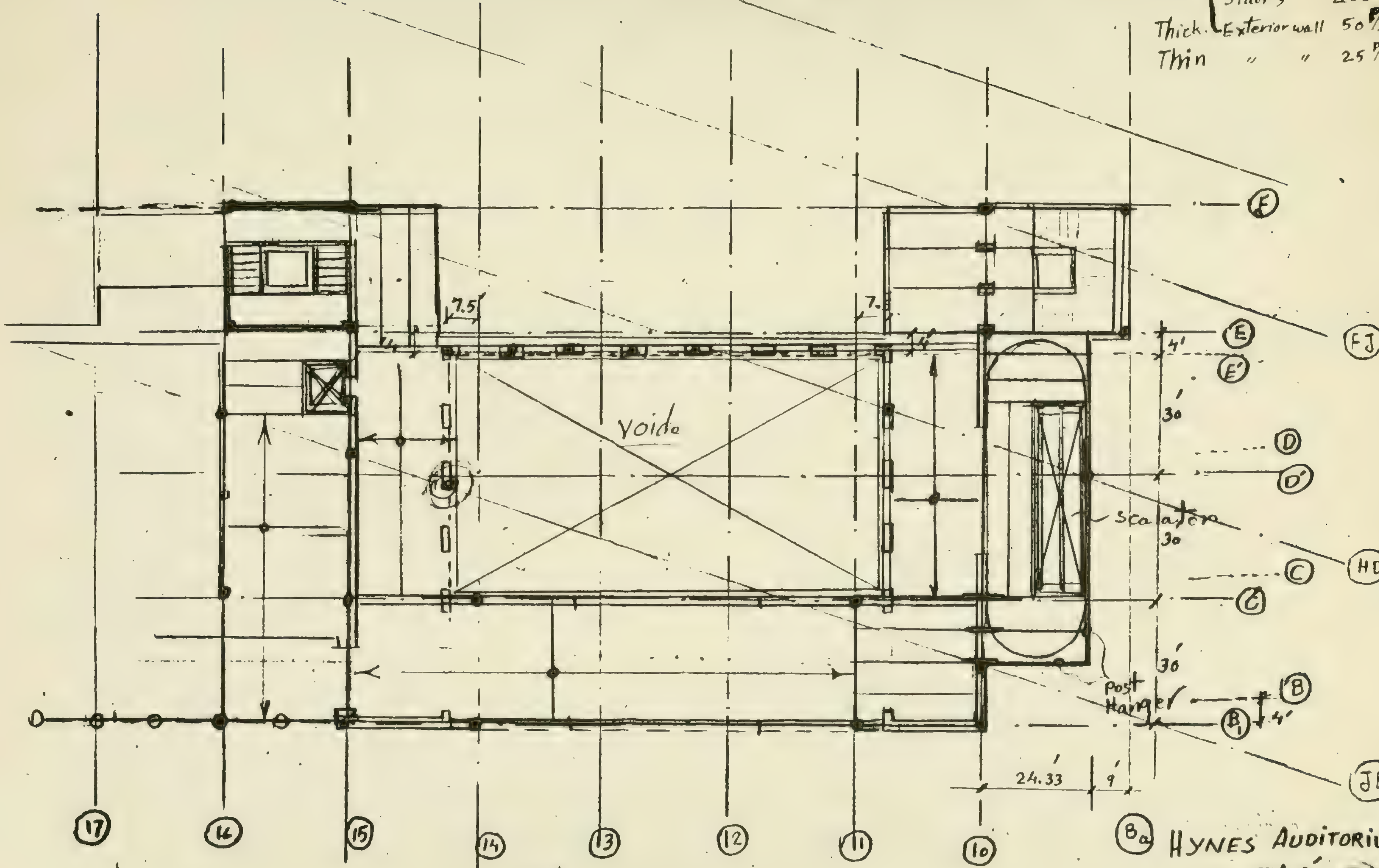
166



8a. HYNES AUDITORIUM
El. 74.83

as Code and request

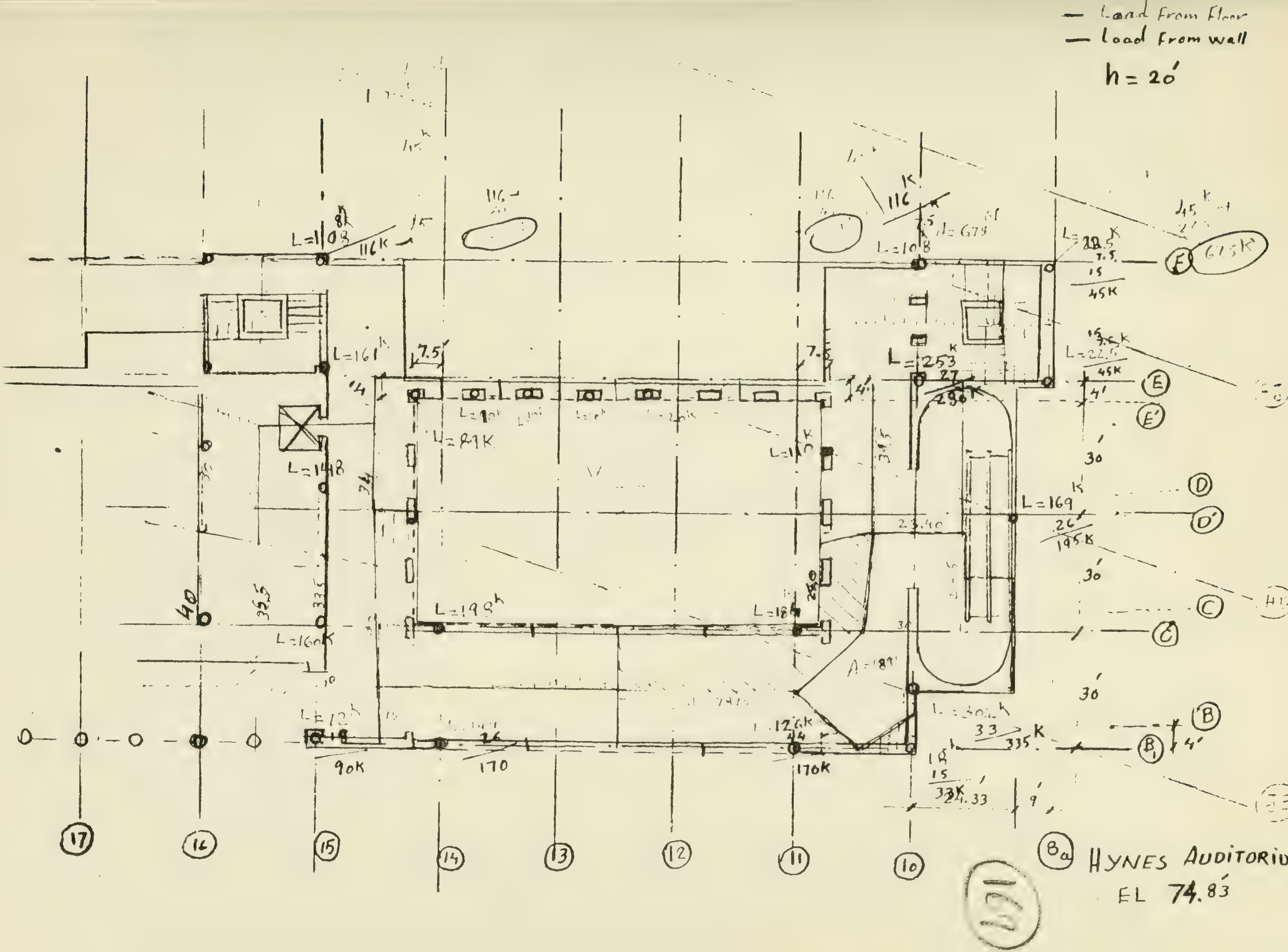
	Floor TL	160 P/SF
	Scalator	250
	Stairs	200
Thick	Exterior wall	50 P/SF
Thin	" "	25 P/SF



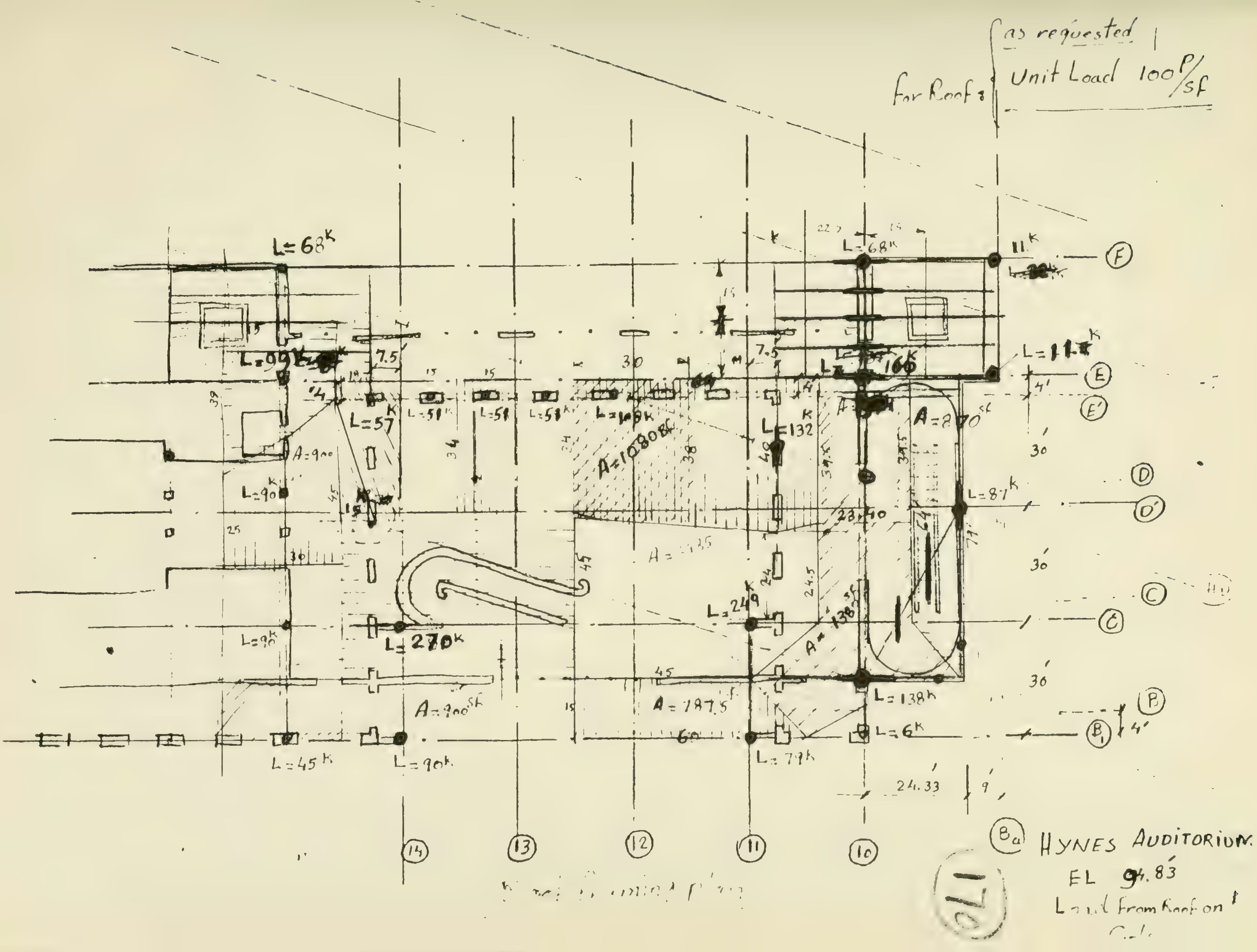
HYNES AUDITORIUM
EL 74.83'

Third Floor framing

168



— Load from Floor
— Load from wall
h = 20'
L = 108
L = 253
27
296 K
L = 169 K
26
195 K
L = 161 K
7.5'
L = 148
L = 160 K
L = 198 K
L = 184
L = 126 K
170 K
170
90 K
116 K
45 K
675 K
33
335 K
24.33
30'
4'
9'
40
HYNES AUDITORIUM
EL 74.83'


for Roof:
as requested
Unit Load 100 P/sf
L = 68K
L = 99K
L = 57K
L = 51K
L = 90K
L = 270K
A = 900 sf
L = 45K
L = 90K
A = 1080 sf
L = 132K
L = 166
A = 270 sf
L = 87K
L = 249
A = 187.5
L = 79K
L = 138K
L = 6K
A = 138 sf
24.33
HYNES AUDITORIUM
EL 94.83
170

Col 12 - (H) G 29 663

G 226 36 1472 –

G 236 left [illegible] 39 [illegible]

G 236 right [illegible] 1433 k

5 piles x 240 x 1.2 = 1440 ≈ 1433

Exist. auditorium wall 1749 - 310 x 30 x 4 =

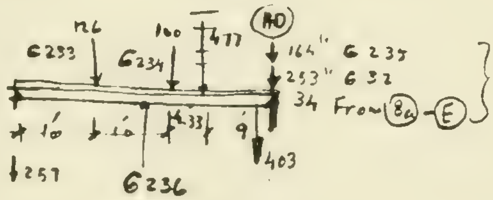
[illegible] weight on the floor [illegible]

[illegible]

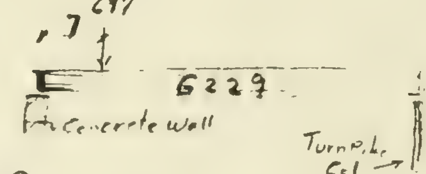
Hynes - Auditorium Extenssion's Cols load.

172

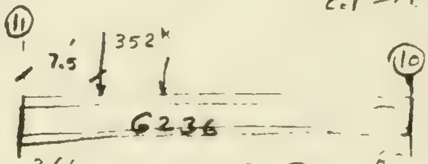
	Roof.	3rd Floor	2nd Floor	1st Floor	Total.
Col. (10)-(JE) (on Concrete wall)	138 K	335 K	335 K	126 K (G232)	934 K
Col. (9a)-(D')	87 K	195	195	—	477 K
Col. (8a)-(HD)					✓ 897 K
Col. (11)-(C')	249	184	184		
Col. (10a)-(HD)	132 K	116	110		
Col. (8a) (E)	11 K	45	58.5		
Col. (10) (E)	160	280	310		



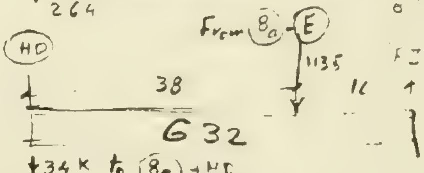
617 K



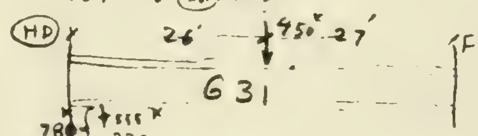
352 K



113.5 K



750 K - 302 K Exist = 448

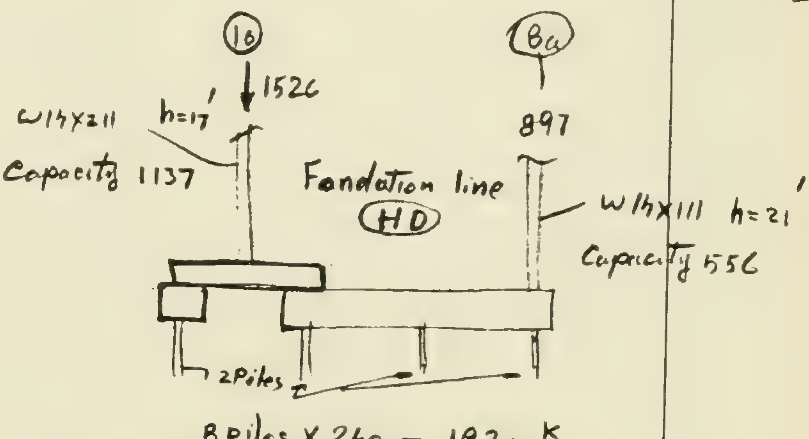


Col. (10) (HD) From G236 Right + G236 Left + G31 + G232
257 + + 403 + 780 + 86 = 1526 K

Col. (11) (HD) From G236 Right G836 Left G229 G30
264 + 339 + 379 + 86 + 596 = 1664 K > 1440 K

Piles capacity
5 x 240 = 1200
1200 x 1.2 = 1440

1526 +
897
2423



2423 K > 2304 K

8 piles x 240 = 1920 K

1920 x 1,20 = 2304 K

WEIDLINGER ASSOCIATES – CONSULTING ENGINEERS   NEW YORK, N. Y.   CAMBRIDGE, MASS.   DATE   BY M.E.

Hynes. A. (Expansion) Estimate of load on T. P. Footing   NO. 1 OF

Loads :

Roof — slab {deck, plywood, Thermal insul., water proof, gravel, Beams} 50 p/sf; Ceiling + Mech. 15; Snow. 30 } = 95 p/sf.

Roof	slab (deck, plywood, Thermal insul., water proof, gravel, Beams)	50 p/sf.	= 95 p/sf.
	Ceiling + Mech.	15	
	Snow.	30	

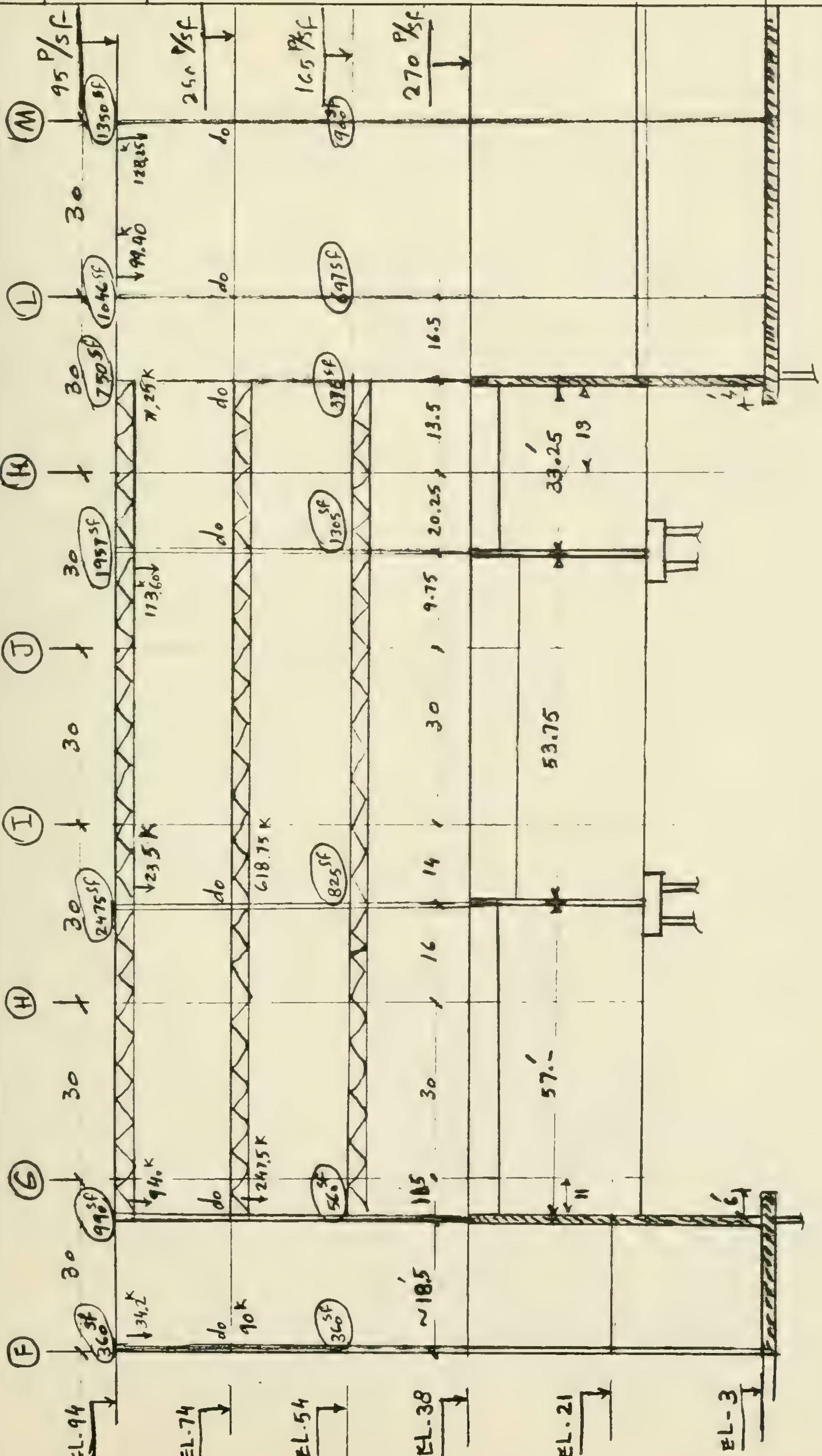
Mech. Room. Floor

slab.	100	250 p/sf
live load	≈ 150	

Typ. Floor	slab	65	165 p/sf
	Live load	100	

Commercial Level.

Exist. floor slab.	5.5" Conc.	60	170	270 p/sf.
	4" flooring	48		
	Beam and other	12		
	Turn. P. Ceiling	50		
live load			100	

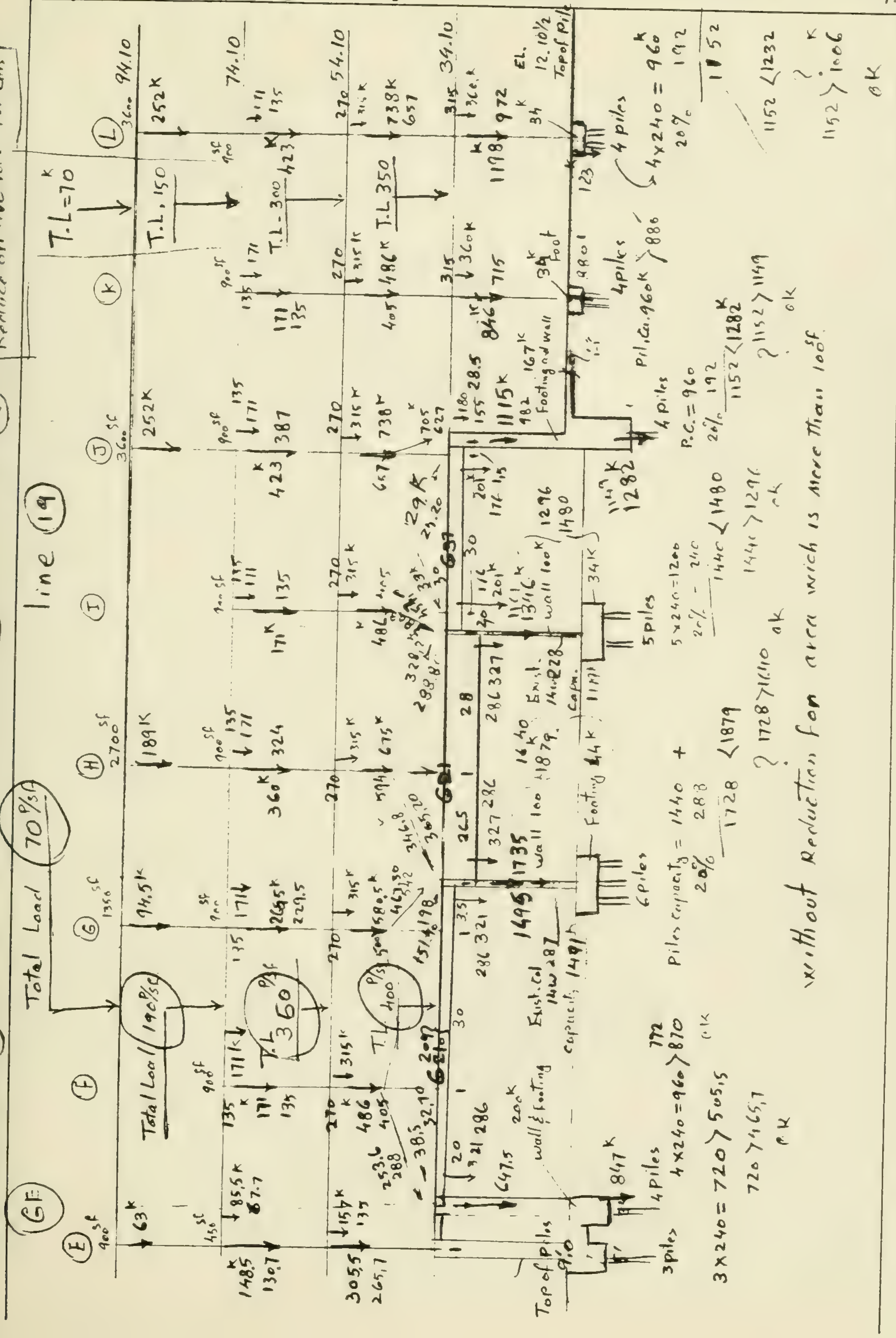

EL.94
EL.74
EL.54
EL.38
EL.21
EL-3
F
G
H
I
J
K
L
M
95 P/SF
16.5 P/SF
270 P/SF
53.75
57.'-
33.25
~18.5

WEIDLINGER ASSOCIATES – CONSULTING ENGINEERS NEW YORK, N. Y. CAMBRIDGE, MASS.

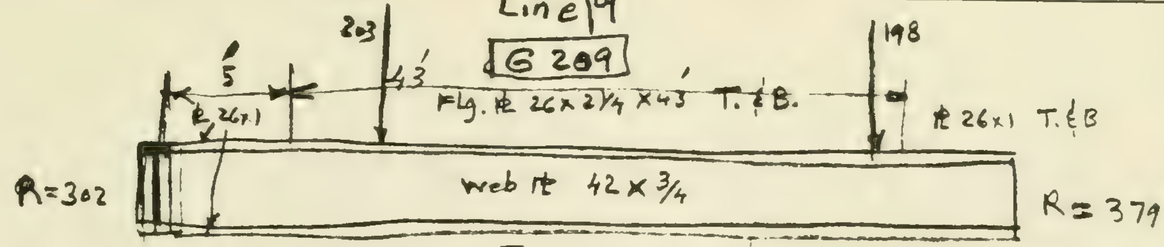
DATE 1/ BY M.E NO. OF

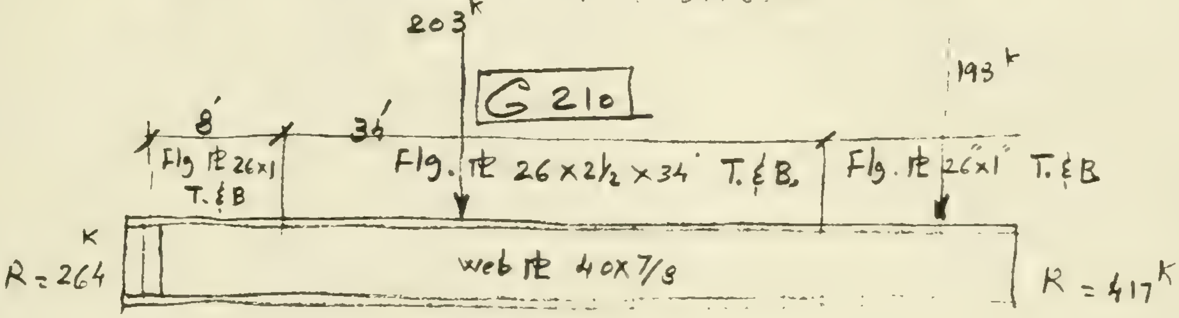
Hynes Au. Checking Capacity of line (19) for Conference Room at Top

175

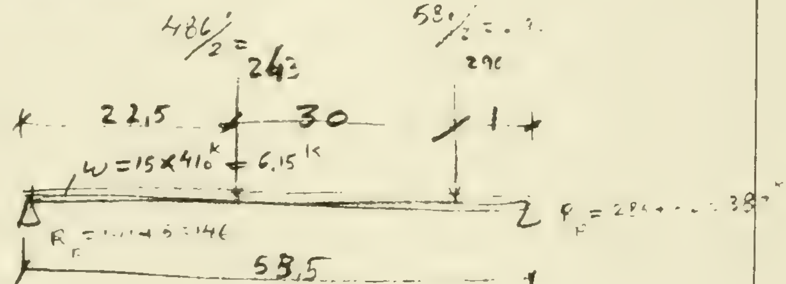


Line 9





Check G 210

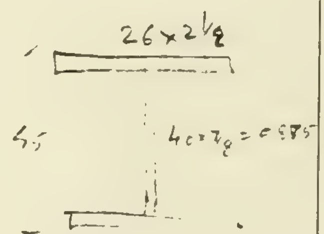


$$M_w = \frac{6.15 \times 53.5^2}{8} = 2200$$

$$M_p = 146 \times \frac{53.5}{2} - 243 \times 5 = 2690 \text{ F.k}$$

$$\Big\} \; 4890 \text{ F.k}$$

$$I = \frac{26\,(45^3 - 40^3)}{12} + \frac{0.875 \times 40^3}{12} = 63\,436$$

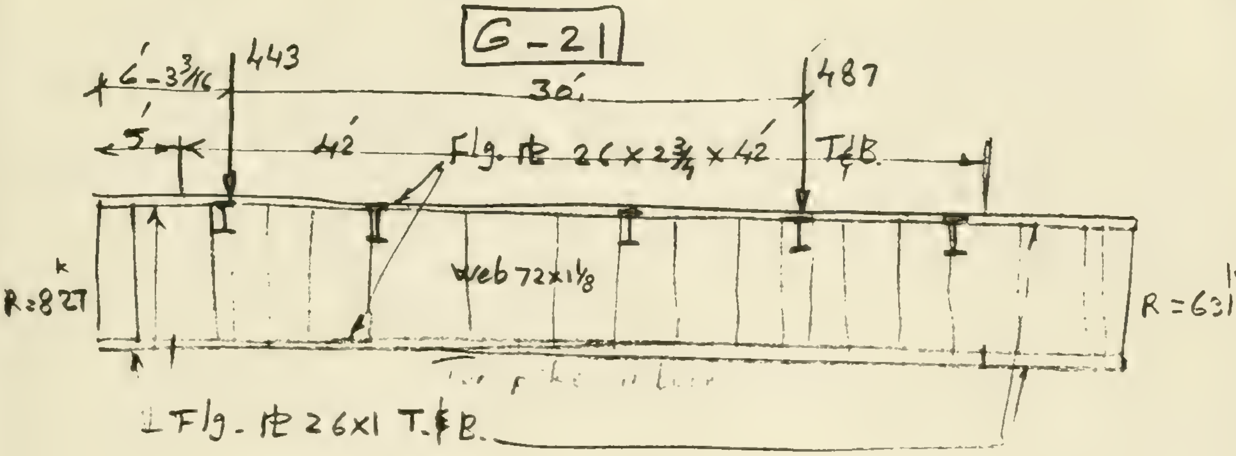


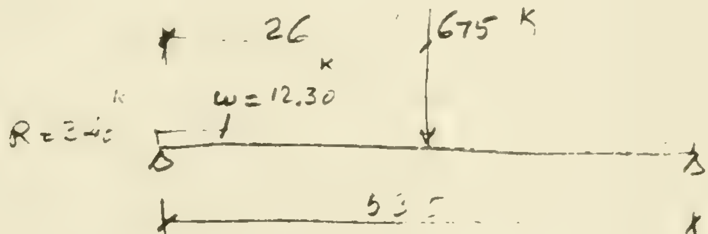
$$S = \frac{63436}{22.5} = 2819 \text{ in}^3$$

$$f_a = \frac{4890 \times 12}{2819} = 12 < 20 \ \text{k/in}^2 \ \text{ok}$$

Line 19

$\boxed{G-21}$



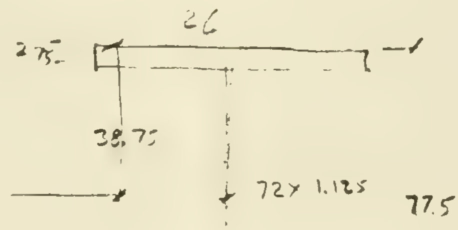


$M_p = 340 \times 26.75 = 9095$ F.k.

$M_w = \dfrac{12.30 \times 53.5^2}{8} = 4400$ F.k

13495 F.k

$$I = \frac{\overset{199845}{26(77.5^3 - 72^3)}}{12} + \frac{\overset{34992}{1.125 \times 72^3}}{12} = 234837$$

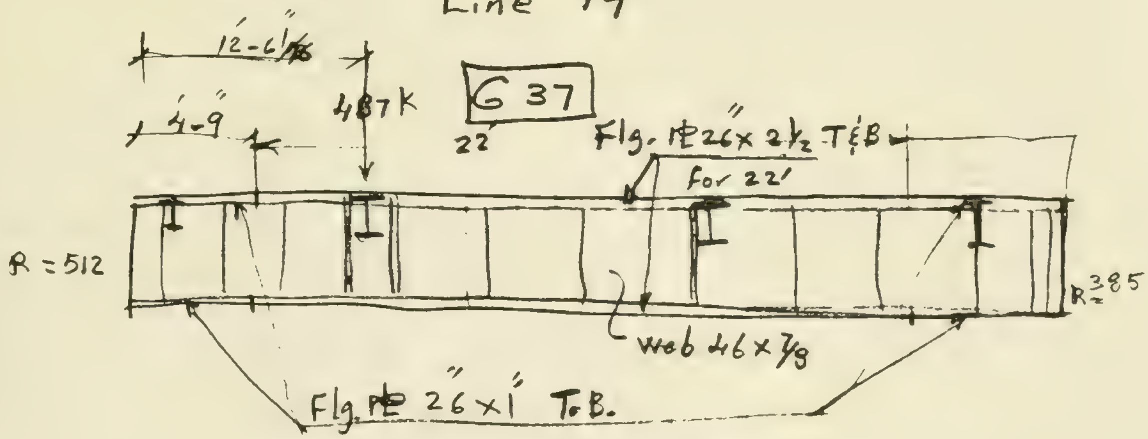


$S = 6060$

$$\frac{13495 \times 12}{6060} = 26.72 > 20$$

(To be Reinforced)

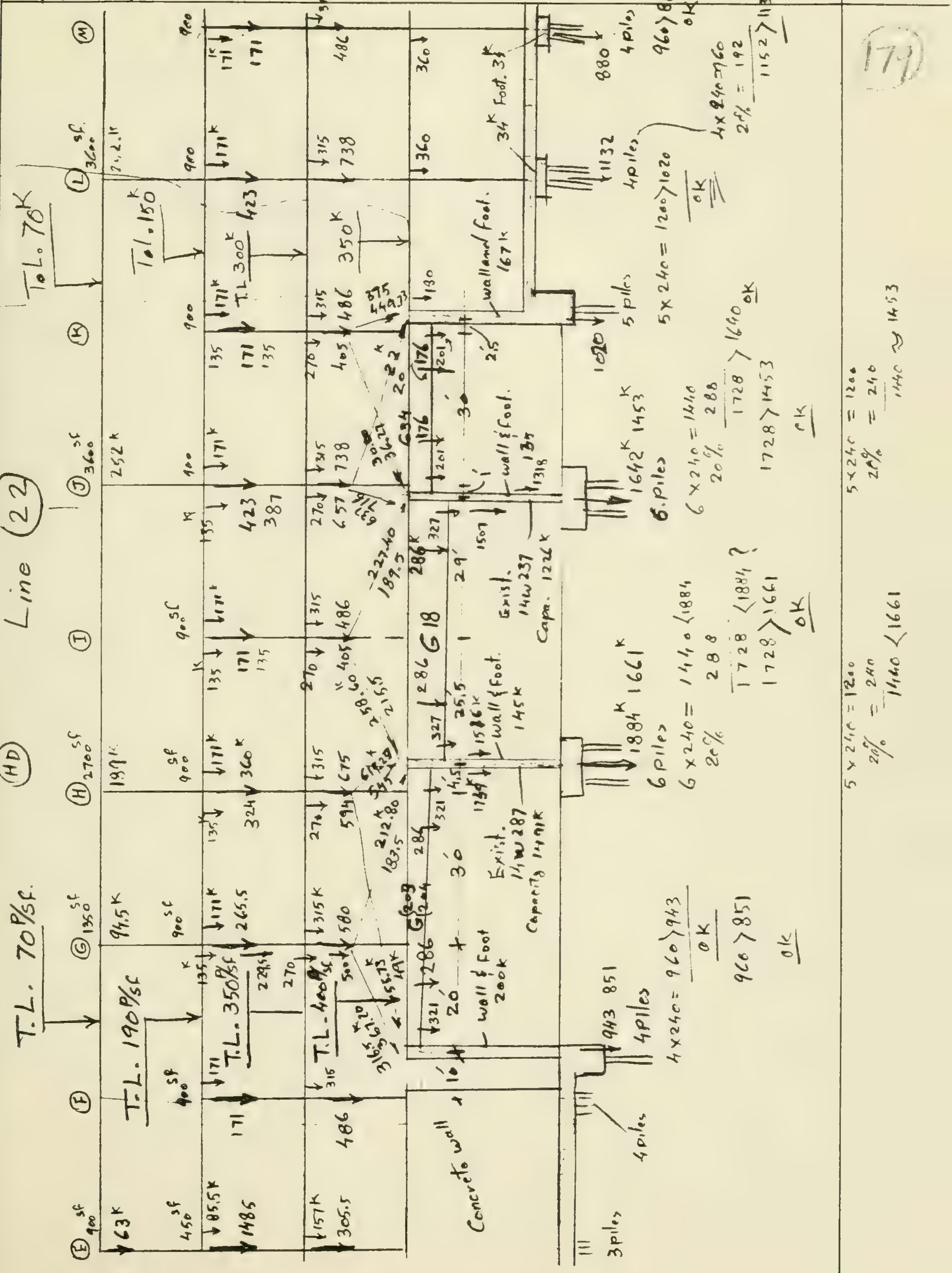
Line 19


12'-6 1/16"
4'-9"
487k
G 37
22'
Flg. PL 26"x 2 1/2" T&B
for 22'
R = 512
R = 385
web 46 x 7/8
Flg. PL 26"x1" T&B.

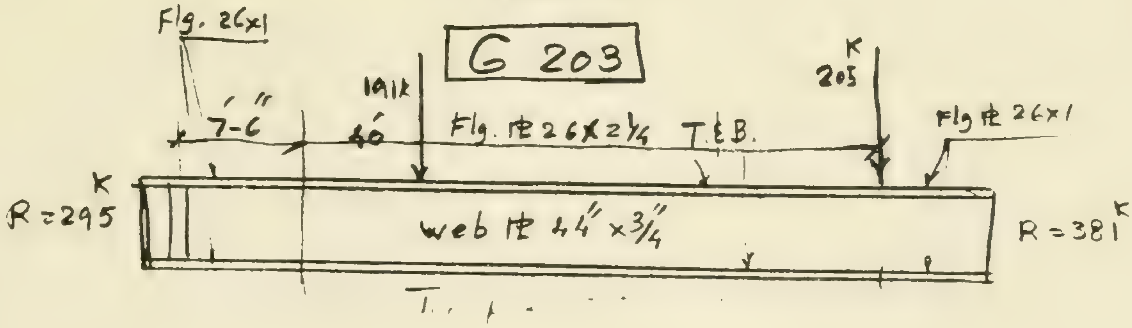
WEIDLINGER ASSOCIATES – CONSULTING ENGINEERS NEW YORK, N. Y. CAMBRIDGE, MASS.

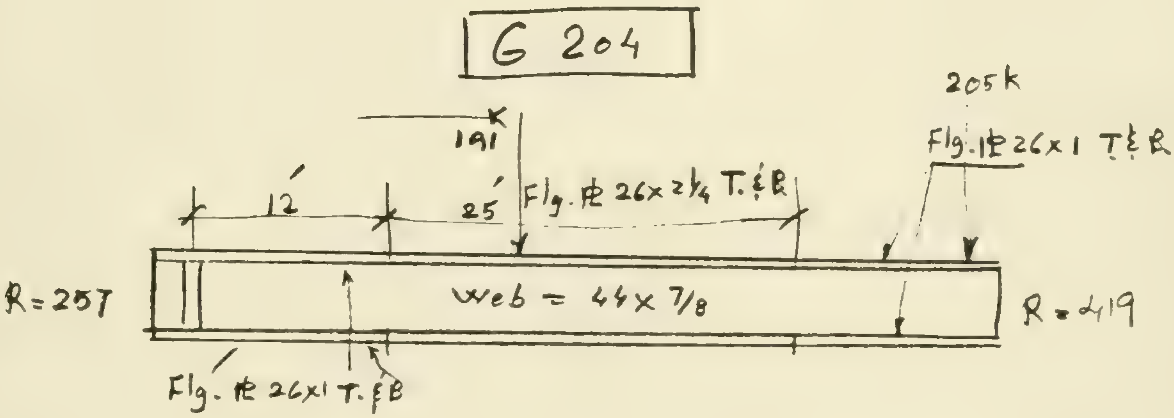
DATE Oct/1/8
BY M.B
NO.
OF

Hynes Au. Checking the capacity of line (22) for Conference Room at Top Level



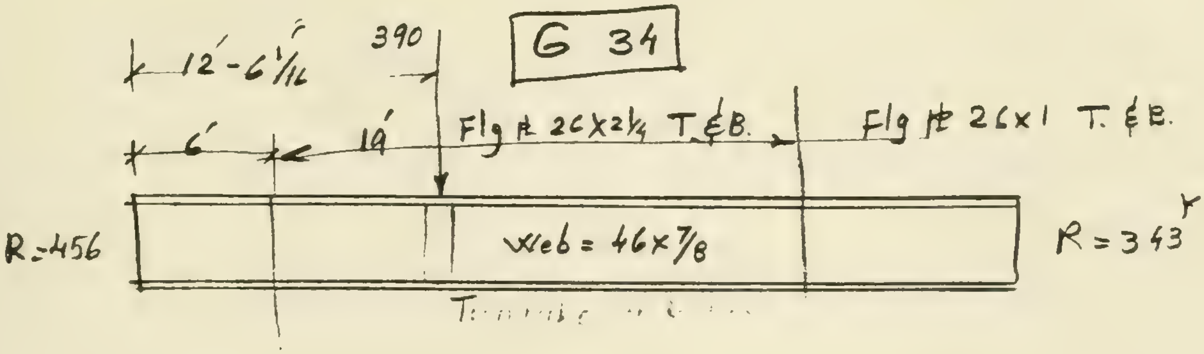
Line 22


Flg. 26x1
G 203
191k
203k
7'-6"
40'
Flg. ℞ 26x2¼ T.&B.
Flg ℞ 26x1
R=295k
web ℞ 44"x¾"
R=381k

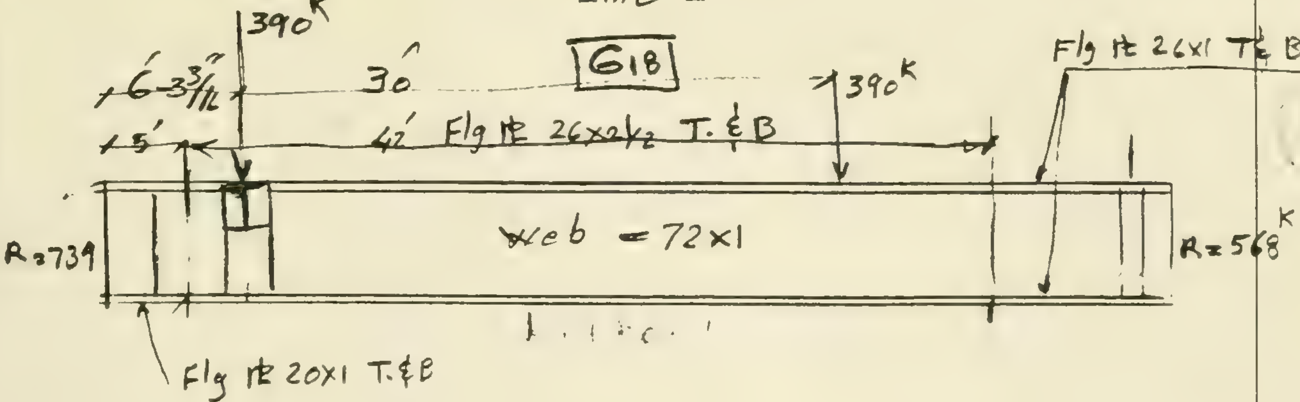

G 204
205k
191k
Flg. ℞ 26x1 T&B
12'
25'
Flg. ℞ 26x2¼ T.&B
R=257
web = 44x 7/8
R=419
Flg. ℞ 26x1 T.&B

Line 22

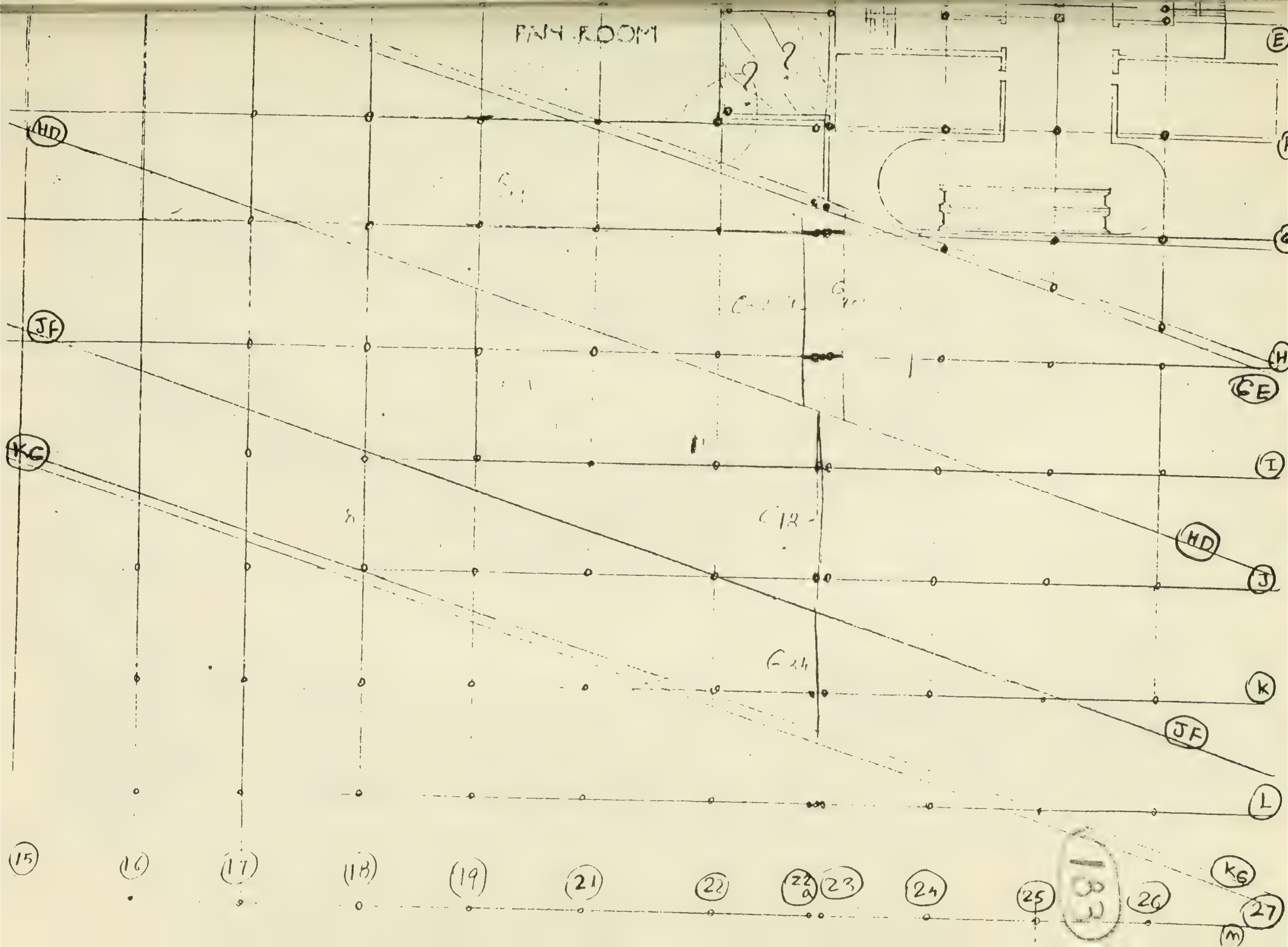
$\boxed{G\ 34}$



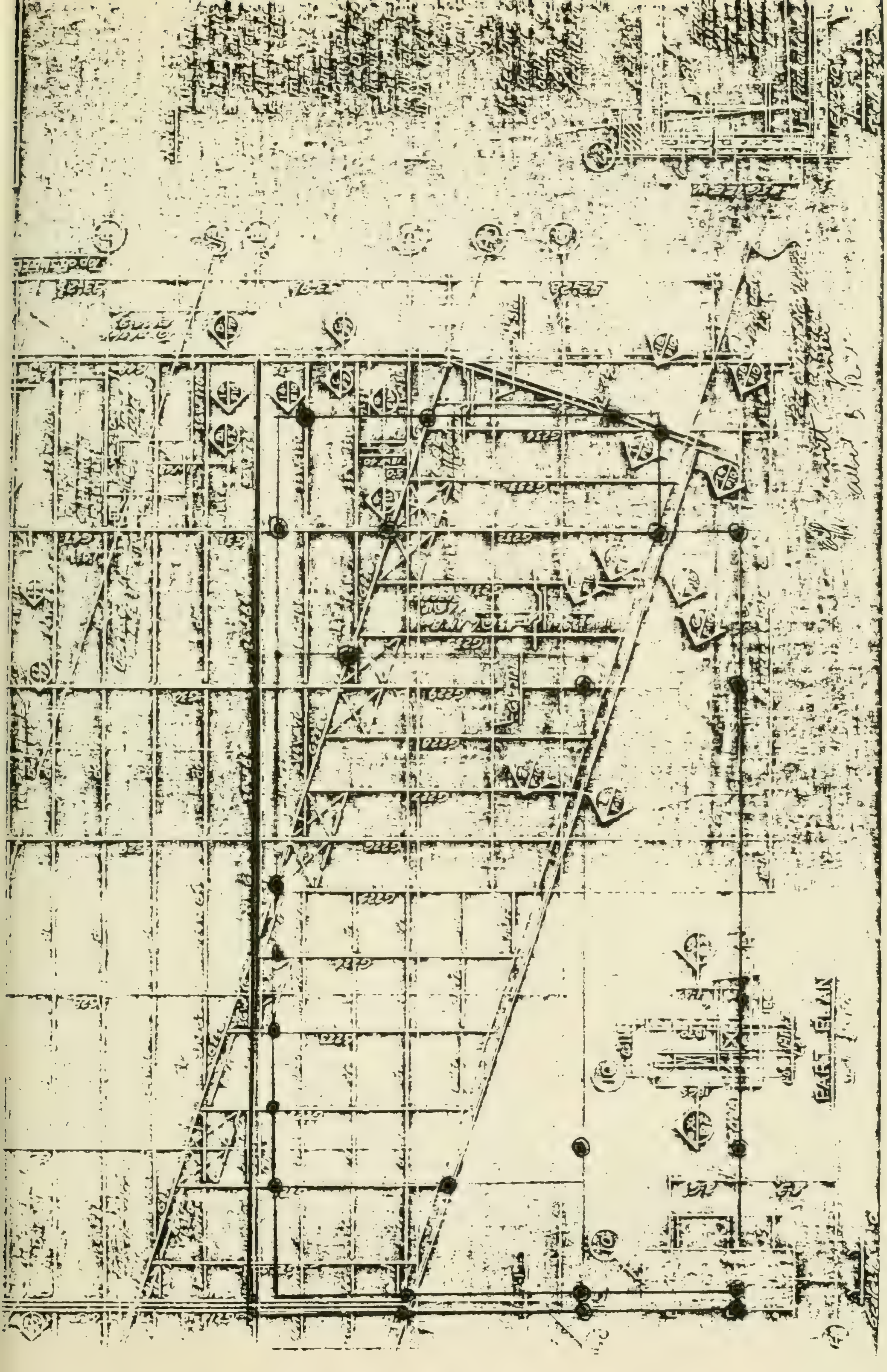
Line 22

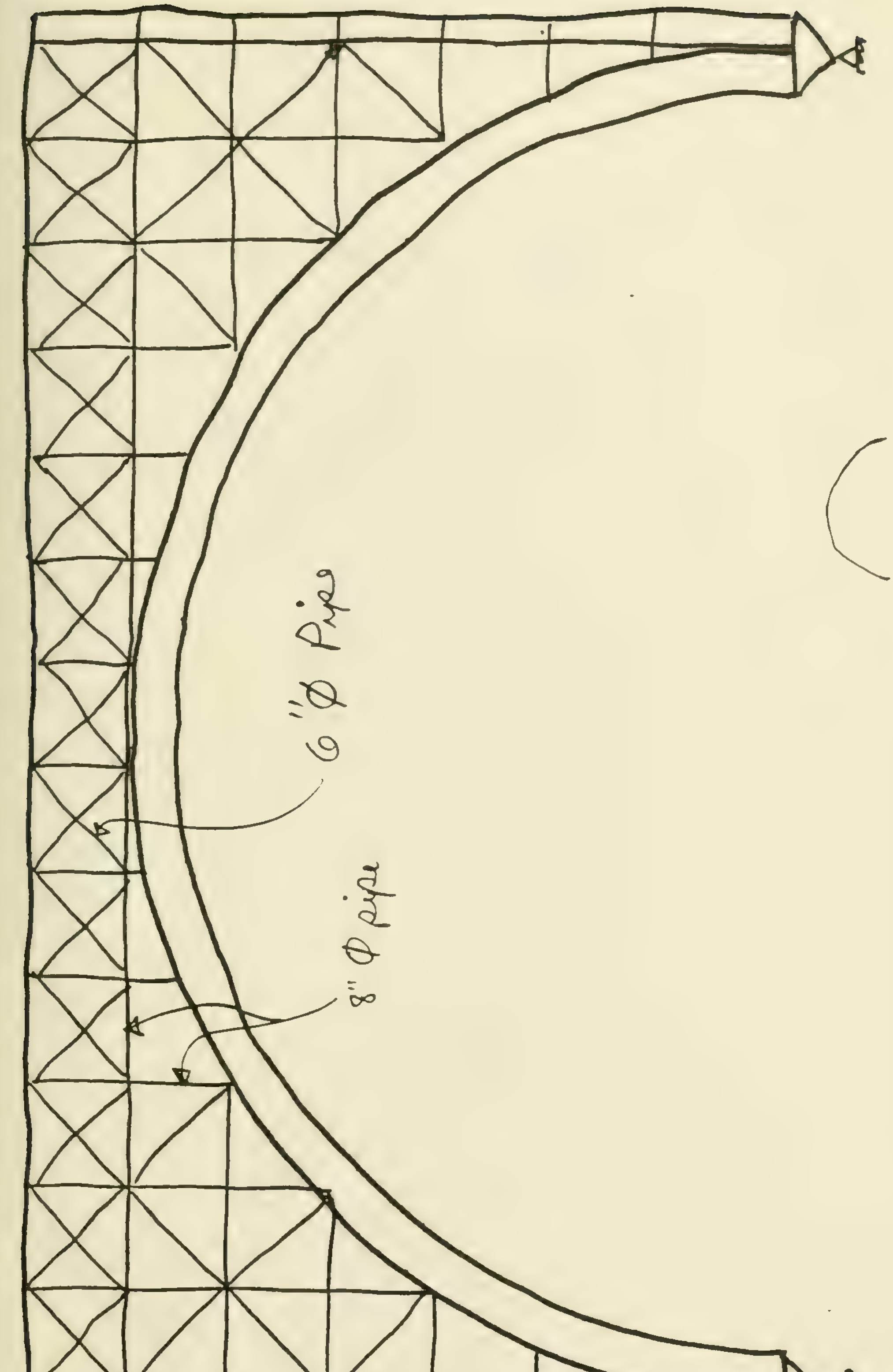

390^K
6'-3 3/16"
30'
G18
390^K
Flg PL 26x1 T&B
5'
42' Flg PL 26x2 1/2 T.&B
web = 72x1
R=739
R=568^K
Flg PL 20x1 T.&B

182



PINK ROOM
E
H
GE
I
HD
J
K
JF
L
KG
27
M
HD
JF
KG
15
16
17
18
19
21
22
22a
23
24
25
26
C 12
C 24


PART PLAN


6" Φ Pipe
8" Φ pipe